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AMERICAN
CONCRETE INSTITUTE

PROCEEDINGS
OF THE
FOURTEENTH ANNUAL CONVENTION

Held at Atlantic City, N. J.,
June 27, 28 and 29, 1918

Volume XIV

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1918

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ROBERT W. LESLEY,
M. M. UPSON,
L. C. WASON.

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CASS GILBERT,
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MILTON D. MORRILL,
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JOHN T. SIMPSON,
K. H. TALBOT,
A. D. WHIPPLE.

PAST OFFICERS.

President.	1905	John P. Given. (Presiding Officer First Convention.)
	1905–14	Richard L. Humphrey.
	1915–16	L. C. Wason.
	1917	W. K. Hatt.
First Vice-President.	1905	A. L. Goetzmann.
	1906–9	Merrill Watson.
	1909–11–12	Edward D. Boyer.
	1913–14	A. N. Talbot.
	1915–16	William K. Hatt.
	1917	Henry C. Turner.
Second Vice-President.	1905–6	John H. Fellows.
	1907–10	M. S. Daniels.
	1911–12	A. N. Talbot.
	1913–14	L. C. Wason.
	1915–16	Henry C. Turner.
	1917	Sanford E. Thompson.
Third Vice-President.	1905	H. C. Quinn.
	1906–7	O. U. Miracle.
	1908	S. B. Newberry.
	1909–12	E. S. Larned.
Fourth Vice-President.	1905–7	A. Monsted.
	1908–9	George C. Walters.
	1909–10	F. A. Norris.
	1911–12	Ira H. Woolson.
Treasurer.	1905	A. S. J. Gammon.
	1906–12	H. C. Turner.
	1913	Henry H. Quimby.
	1914	Robert A. Cummings.
	1915–17	Robert W. Lesley.
Secretary.	1905–6	Charles C. Brown.
	1907	W. W. Curtis.
	1908–9	George C. Wright.
	1910	Edward E. Krauss (Acting).
	1911–14	Edward E. Krauss.
	1915	Charles L. Fish.
	1915	J. M. Goodell (Acting).
	1916	Harold D. Hynds.
	1917	Henry B. Alvord.

Directors.	1913	Benjamin F. Affleck.
		William P. Anderson.
		Edward D. Boyer.
		W. L. Church.
		Charles Derleth, Jr.
		Ernest L. Ransome.
	1914	Benjamin F. Afflck.
		William P. Anderson.
		Edward D. Boyer.
		W. L. Church.
		John B. Leonard.
		Robert W. Lesley.
	1915	William P. Anderson.
		Ernest Ashton.
		Edward D. Boyer.
		William H. Ham.
		John B. Leonard.
		Alfred E. Lindau.
	1916	William P. Anderson.
		Ernest Ashton.
		Edward D. Boyer.
		Charles A. Gow.
		Alfred E. Lindau.
		John G. Treanor.
	1917	William P. Anderson.
		Ernest Ashton.
		Edward D. Boyer.
		Charles R. Gow.
		Ernest McCullough.
		John G. Treanor.

BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next

Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candicates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

Article III.

MEETINGS.

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

Sec. 2. The annual dues of each member shall be ten dollars ($10.00).

Sec. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

SECTION 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

ARTICLE VI.

AMENDMENT.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE FOURTEENTH ANNUAL CONVENTION.

Hotel Traymore, Atlantic City, N. J.

First Session, Thursday, June 27, 1918, 8 p. m.

The Convention, in joint session with the American Society for Testing Materials, was called to order by S. S. Voorhees, Vice-President of the American Society for Testing Materials.

Robert W. Lesley in the chair.

The report of the A.S.T.M. Committee C-1 on Cement was presented by its Chairman, R. S. Greenman.

The report of the A.S.T.M. Committee C-2 on Reinforced Concrete was presented by its Secretary, Richard L. Humphrey.

The report of the A.C.I. Committee on Treatment of Concrete Surfaces was presented by its Chairman, J. C. Pearson. The report was accepted and ordered printed.

The A.C.I. paper, "Tests of Stucco," was read by its author, J. C. Pearson.

The A.C.I. paper, "Fire Tests of Concrete Columns," was read by its author, W. A. Hull.

The A.S.T.M. paper, "Effect of Age on the Strength of Concrete," was read by its author, D. A. Abrams.

The A.S.T.M. paper, "Proportioning the Materials of Mortars and Concretes by Surface Areas of Aggregates," was read by its author, Capt. L. N. Edwards.

Second Session, Friday, June 28, 1918, 10.30 a. m.

President W. K. Hatt in the chair.

Frank C. Wight in the chair.

The paper, "Apparatus for Testing Under Uniform Load," was read by Prof. W. K. Hatt in the absence of its author, H. H. Scofield.

President W. K. Hatt in the chair.

The report of the Committee on Building Blocks and Cement Products was presented by the Secretary of the Institute

The following papers were read and discussed:

> "Ornamental and Decorative Concrete," by R. F. Havlik. Read by the Secretary of the Institute in the absence of the author.
>
> "Problems in Devising a New Finish for Concrete," by J. J. Earley.

"Reinforced-Concrete Products for Railway Purposes," by Charles Gilman."
"Reinforced-Concrete Pressure Pipe," by Coleman Meriwether.
"Tests of Blast Furnace Slag as Concrete Aggregate," by P. J. Freeman.

THIRD SESSION, FRIDAY, JUNE 28, 1918, 3 P. M.

President W. K. Hatt in the chair.

The report of the Committee on Reinforced-Concrete and Building Laws was read by the Secretary of the Institute. The report was laid on the table.

The following papers were read and discussed.

"Some Tests on the Effect of Age and Condition of Storage of Concrete," by H. F. Gonnerman. Read by A. N. Talbot in the absence of the author.
"Moment Coefficients in Flat-Slab Design," by W. K. Hatt
"Tests of an Eight-year Old Flat-Slab Floor of the Western Newspaper Building," by A. N .Talbot and H. F. Gonnerman. Read by A. N. Talbot.
"Theory and Test of Flat-Slab with Ring Reinforcement," by Edward Smulski.
"Design of Concrete Chimneys," by J. G. Mingle.

FOURTH SESSION, FRIDAY, JUNE 28, 1918, 8 P. M.

President W. K. Hatt in the Chair.

The following papers were read and discussed:

"Industrial Housing and Labor Turnover," by Leslie H. Allen.
"Advantages and Disadvantages of the Concrete House," by John E. Conzelman. Read by the Secretary of the Institute in the absence of the author.
"Recent Examples of Concrete Buildings in Housing Work," by A. B. Whipple.
"Methods of Construction of Concrete Houses," by Lieut. K. H. Talbot. Read by L. H. Allen in the absence of the author.
"Architectural Design of Concrete Houses," by Emile G. Perrot. Read by title in the absence of the author.
"Interior Design of Concrete Houses," by Milton D. Morrill.

The report of the Committee on Industrial Concrete Housing was presented by its chairman, Leslie H. Allen.

The representative of the Institute on the Joint Committee on Fireproofing, A. E. Lindau, reported that the committee had published a specification for fire tests. On motion the specification was accepted by the Institute and ordered printed in the Proceedings with the statement that it is the purpose of the Institute to make it a standard specification when it can take its usual course prescribed by the by-laws for the adoption of standards.

FIFTH SESSION, SATURDAY, JUNE 29, 1918, 10 A. M.

President W. K. Hatt in the chair.

The report of the Committee on the Far Rockaway Fire was presented by its Chairman, Richard L. Humphrey. The report was accepted and ordered printed.

The report of the Committee on Concrete Roads and Pavements was presented by its Chairman, H. E. Breed. Revisions of specifications standards Nos. 5, 17, 18, 19 and 20 were sent to letter ballot for adoption as standards. Remainder of report accepted and ordered printed.

The following papers were read and discussed:

> "Concrete Roadways for the Industrial Plant," by G. S. Eaton.
>
> "Effect of Time of Mixing on the Strength and Wearing Qualities of Concrete," by D. A. Abrams.

The report of the Committee on Sidewalks and Floors was presented by its Chairman, J. E. Freeman. Report accepted and ordered printed.

The following papers were read and discussed:

> "Distortions and Vertical Changes of Concrete Pavement Slabs Due to Subgrade Movements," by J. W. Lowell.
>
> "How to Get the Best Surface on a Concrete Road," by A. H. Hunter. Read by title in the absence of the author.
>
> "Some Experiences on the Vermilion County Concrete Roads," by P. C. McArdle. Read by title in the absence of the author.

The report of the Committee on Plain and Reinforced-Concrete Sewers was presented by Coleman Meriwether of the committee. Report with suggested specifications referred back to the committee and ordered not printed in the Proceedings.

SIXTH SESSION, SATURDAY, JUNE 29, 1918, 3 P. M.

President W. K. Hatt in the chair.

Business Session.

The Secretary reported that the letter ballot had resulted in the election of the following officers for the ensuing year:

President: William K. Hatt.
Vice-President: Henry C. Turner.
Treasurer: Robert W. Lesley.
Directors: *First District*—C. R. Gow.
Second District—E. D. Boyer.
Sixth District—J. G. Treanor.

The Treasurer read the report of the Treasurer and of the Auditors.

The Secretary read the annual report of the Board of Direction.

On motion the following resolution was adopted:

"*Resolved,* That the American Concrete Institute in convention assembled learns with regret of the death of Dr. Edgar Marburg, who for so many years has so faithfully served as Secretary of the American Society for Testing Materials, and desires to extend its sympathy to the family and our sister society, and that a copy of this resolution be entered on the minutes of this Institute, sending one to the American Society for Testing Materials and one to the family."

The following papers were read and discussed:

- "Reinforced-Concrete Columns under Eccentric Load," by L. J. Mensch. Read by title in the absence of the author.
- "Reinforced-Concrete Cold Storage Warehouse at Boston," by G. H. Brazer. Read by title in the absence of the author.
- "Core Construction for Reinforced-Concrete Floors," by A. H. Bromley, Jr.
- "Relation of Costs to Design of Reinforced-Concrete Buildings," by C. W. Mayers.
- "Construction of C. & N. W. Ry. Grain Elevator," by F. C. Huffman. Read by title in the absence of the author.
- "Construction of Reinforced-Concrete Building for American Can Co.," by N. M. Loney. Read by W. K. Hatt in absence of the author.
- "Reinforced-Concrete Flat-Slab Railway Bridges," by A. B. Cohen.

The report of the Committee on Research was presented by its Chairman, W. A. Slater. The report, which was one of progress, was accepted by the Convention.

SEVENTH SESSION, SATURDAY, JUNE 29, 1918, 8 P. M.

Leonard C. Wason in the chair.

The report of the Committee on Nomenclature was presented by its chairman, W. A. Slater. The report was accepted by the Convention.

The report of the Committee on Concrete Barges and Ships was presented by its chairman, H. C. Turner.

The following papers were read and discussed:

"Plasticity and Temperature Deformations in Concrete," by S. C. Hollister.

"Developments in Concrete Barges and Ships," by J. E. Freeman.

"Problems Arising in the Design and Construction of Reinforced-Concrete Ships," by R. J. Wig and S. C. Hollister. Read by R. J. Wig.

"The Yard for Building Concrete Ships at Wilmington, N. C.," by A. G. Monks. Read by title in the absence of the author.

"Design and Construction of a Concrete Barge," by L. L. Livingston.

President W. K. Hatt in the chair.

The President declared the Convention adjourned *sine die.*

Papers Read Before the 14th Annual Convention of the American Concrete Institute

EFFECT OF TIME OF MIXING ON THE STRENGTH AND WEAR OF CONCRETE.

By Duff A. Abrams.*

Introduction.

Machine-mixing has almost entirely replaced hand-mixing of concrete on work requiring the use of large quantities of material. While it is generally recognized that good results can be secured by hand-mixing it is also well known that this result can be accomplished only at the cost of conscientious effort and much hard work. It has long been known that the thoroughness of mixing, whether by machine or hand, has an important influence on the quality of the finished concrete. Reference may be found in engineering literature to several series of tests in which the effect of duration of mixing was studied. So far as hand-mixing is concerned such studies have been generally confined to briquet tests made on cement mortars. The tests on machine-mixed concrete which have been reported covered such a narrow range that no quantitative measure of the effect of continued mixing has been developed. No sustained effort has been made, so far as the writer is aware, to analyze the factors which enter into the beneficial effects of continued mixing. Due to a more or less vague impression of the value of longer mixing time, there has been a marked tendency during the past year or two on the part of specification writers toward lengthening the time the batch should remain in the mixer.

There can be no question of the importance of thorough mixing of concrete. Undoubtedly much inferior concrete is produced by undermixing. Undermixing, coupled with the excess of mixing water which is too frequently used, forms a combination which is very conducive to defective concrete or concrete of low strength and wearing resistance. Of the two evils, excess water is by far the greater.

Due to the mutiplicity of sizes and designs of batch mixers the exact basis for the mixing period has not been clear. Should a minimum time limit be specified? Or should a minimum number of revolutions of the drum be required? If a minimum time limit for mixing is specified, what is a proper basis for mixers of different capacities, or mixers operating at different rates? Whatever basis is used for fixing the minimum amount of mixing which would be acceptable, several other questions of equal importance are immediately raised. What is the effect of the time if the proportions of the concrete are

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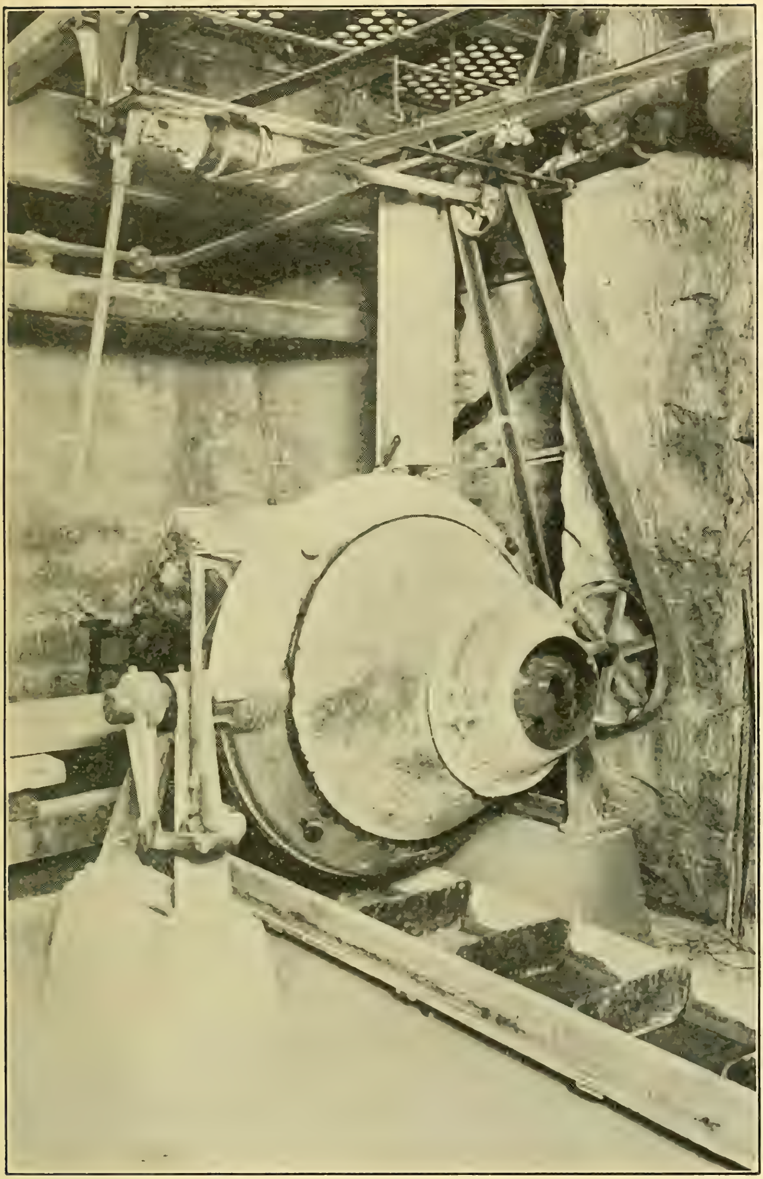

FIG. 1.—3½-CU. FT. CONCRETE MIXER USED IN TESTS.
The trough shown in the foreground was used to receive the concrete as the batch was discharged.

changed? The change of proportions would involve varying the relative quantities of cement, water and aggregate, the size and grading of the aggregates, etc.

It was with a view to securing more definite information on some of these problems that the tests covered in this report were carried out. Two forms of test were used: (1) compression tests of 6 by 12-in. concrete cylinders, and (2) wear tests of concrete blocks, 8 in. square and 5 in. in thickness. The wearing resistance of concrete is of great interest in view of the widespread use of concrete for the construction of roads, pavements, floors, walks, etc. The wear tests were made in the Talbot-Jones rattler.

Over 3800 specimens were made in the experiments covered in this report. The 1-year compression tests have not been made. For the conditions of the tests definite results were secured on the following topics:

1. Influence of quantity of mixing water on the strength and wear.
2. Effect of time of mixing (15 sec. to 10 min.) on the compressive strength.
    - (*a*) Concrete of different consistencies.
    - (*b*) Concrete of different cement content.
    - (*c*) Concrete made of aggregates of widely different size.
3. Effect of time of mixing on the wear of concrete of different consistencies.
4. Relation between wear and compressive strength.
5. Effect of rate of rotation of mixer drum on the strength of concrete.
6. Effect of temperature of mixing water on the strength of concrete.
7. Effect of age on the strength of concrete.
    - (*a*) For different times of mixing.
    - (*b*) For different mixes and consistencies.
    - (*c*) For different sizes of aggregates.
8. Comparison of machine-mixing with method of hand-mixing used in this Laboratory.
9. Uniformity of mixer tests.

These experiments form a part of the researches in the properties of concrete and concrete materials being carried out through the co-operation of the Portland Cement Association and Lewis Institute at the Structural Materials Research Laboratory, located at Lewis Institute, Chicago. The tests must be considered as merely preliminary to any comprehensive investigation of the performance of concrete mixers. Such an investigation would include machines of many types and capacities and numerous other factors which have not been studied. It is hoped that an opportunity will be found for extending these mixer investigations beyond their present limits.

Acknowledgment is due to the Chicago Gravel Company, Chicago, for their courtesy in supplying the aggregates used in these tests and to The T. L. Smith Company, of Milwaukee, who donated one of their 3½-cu. ft. "Mascot" concrete mixers for use in the Laboratory.

Outline of Tests.

For convenience each group of experiments is given a series number. It is believed that confusion will be avoided by referring to the tests by the series numbers. Table 1 gives the titles of the series and the number of tests of each kind.

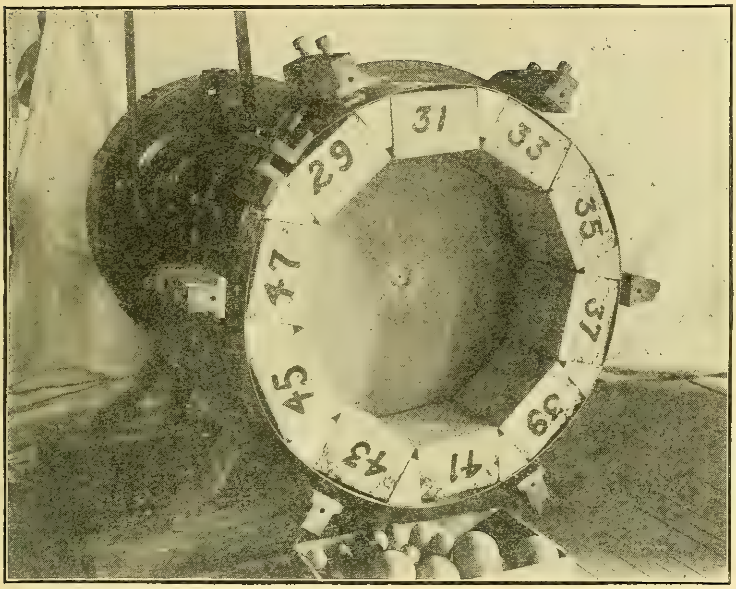

FIG. 2.—TALBOT-JONES RATTLER WITH CONCRETE WEAR BLOCKS IN PLACE.
Wear tests were made on blocks 8 in. square, 5 in. thick.

The tests on effect of time of mixing covered in this report were made on concrete mixed in a tilting mixer with a double-cone drum, capacity 3½ cu. ft., manufactured by The T. L. Smith Company. In all these tests the mixer was loaded to capacity and the entire batch was made into test pieces.

Series 81 was preliminary in nature, carried out for the purpose of studying the variation in the compressive strength of specimens made from different parts of the same batch of machine-mixed concrete. Three batches of different consistencies were mixed. Compression tests on 6 by 12-in. cylinders were made at 7 and 28 days.

Series 89 included a study of the effect of time of mixing, using concretes of consistencies varying from a fairly stiff to a wet mix, the proportion of cement and the grading of aggregates being the same in all tests. Batches of each consistency were mixed for periods ranging from 15 sec. to 10 min. Compression tests were made at ages of 7 and 28 days and 2 mo., and wear tests at 2 mo. This is the only series in which wear tests were made.

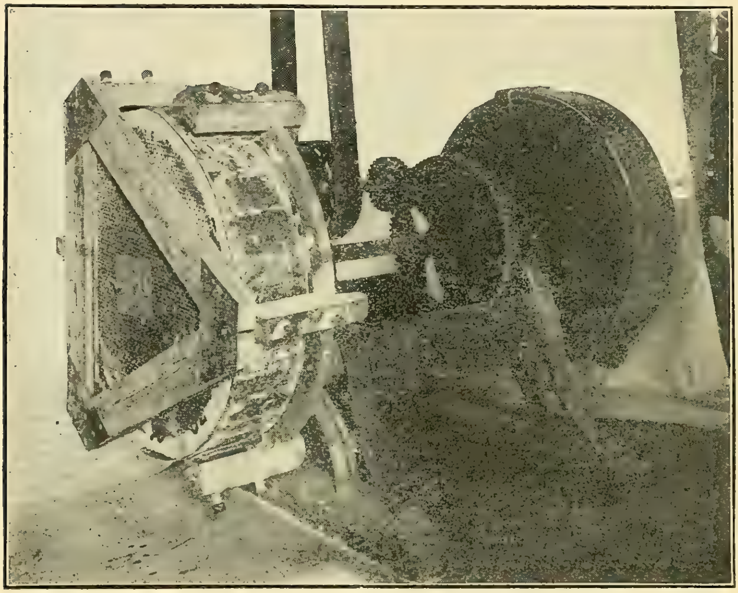

FIG. 3.—TALBOT-JONES RATTLER WITH HEAD CLOSED READY FOR TEST.
The machine was operated for 1800 revolutions at 30 r.p.m.

In Series 93 a study was made of the effect of time of mixing, varying both the mix and the size of aggregate. The first part of the series consisted of variations in the quantity of cement from a 1–15 to a 1–2 mix, using aggregates of the same grading (0–1¼ in.). In the second part of the series a 1–5 mix was used, the aggregate ranging in size from sand all of which passed the 14-mesh sieve to a sand and pebble mixture graded up to 2 in. Batches of each mix, etc., were mixed for periods of 15 sec. to 10 min. Parallel sets of tests on hand-mixed concrete were made for comparison with the machine-mixed specimens in both of the groups mentioned above. In the hand-mixed concrete only one mixing period was used. Compression tests were made at ages of 7 and 28 days and 3 mo. The 1-year tests are not yet due.

Series 96, "Effect of Rate of Rotation of Mixer Drum." Concrete of 1–5 mix, aggregate graded to 1¼ in. in size was mixed for 1 minute at 6 different rates, ranging from 8 to 30 r. p. m. The rate recommended by the manufacturers for this mixer is 18 r. p. m. Concrete of four different consistencies was used for each rate. Compression tests were made at ages of 7 and 28 days and 3 mo. The 1-year tests are not yet due.

Series 97, on effect of temperature of mixing water on the strength of concrete is somewhat foreign to the subject-matter of this report, but since the tests were made at the same time, using the same materials and under similar conditions, it is considered desirable to include the results here.

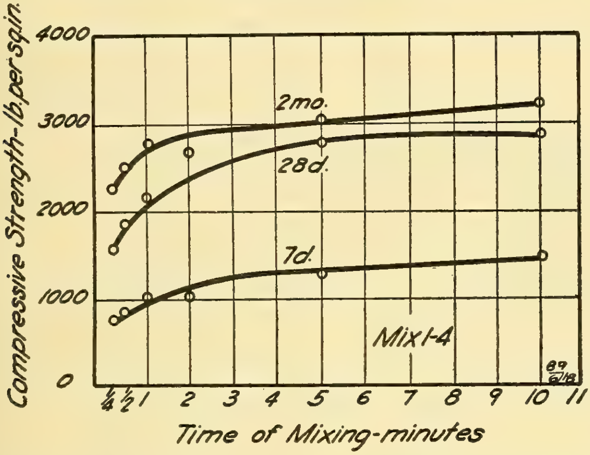

FIG. 4.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 89—Each value is the average of 24 compression tests of 6 by 12-in. cylinders from batches of 6 different consistencies.

Compare Fig. 21, 23, 25, 27, 29.

The temperature of the mixing water was varied at intervals of 10° C. from 0 to 100° C. Four different consistencies were used. Duplicate batches were mixed for each condition on different days. Compression tests were made at ages of 7 and 28 days and 3 months.

MATERIALS.

The cement used consisted of a mixture of equal parts of four brands purchased in the Chicago market. The brands were thoroughly mixed by placing one sack of each in the mixer and running for about 1 minute. Complete tests of the different lots of mixed cements are given in Tables 2 and 3.

The aggregates consisted of sand and pebbles from the Chicago Gravel Company's plant near Elgin, Ill. This material was received in carload lots in the form of coarse torpedo sand, graded up to ⅜ in. in diameter, and pebbles graded from ¼ to 1¼ in. Before using, the sand was screened through a No. 4 sieve. All material retained on this sieve was added to the pebbles. The pebbles were all from the same shipment. The material was shoveled over several times and stored for use in a shallow bin. In most of the tests the aggregate was graded up to 1¼ in. by mixing a certain percentage of sand with the pebbles. In Series 93 the aggregates of the smaller sizes were prepared by using the sand which was finer than No. 14, No. 8 and No. 4 sieves respectively.

Table 1.—Outline of Test Series.

Series No.	Title.	Total Number of Tests.		
		Compression.	Wear.	Miscellaneous.
81	Uniformity of machine-mixed concrete....................	44	...	70
89	A study of the time of mixing concrete (effect of consistency)	612	...	70
93	A study of the time of mixing concrete (effect of mix and size of aggregate)..	1344	240	82
96	Study of rate of rotation of mixer drum..................	576	...	70
97	Effect of temperature of mixing water on the strength of concrete..	660	...	70
	Total..	3236	240	362

Table 2.—Miscellaneous Tests of Cement.

Each cement is composed of a mixture of equal parts of four brands purchased in Chicago market.

Cement Lot No.	Used in Series No.	Fineness Test, Residue on 200.	Normal Consistency, per cent.	Time of Setting.				Soundness Test (over boiling water).
				Vicat Needle.		Gillmore Needle.		
				Initial, h. m.	Final, h. m.	Initial, h. m.	Final, h. m.	
3768......	81	18.0	24.5	5 40	8 40	6 20	8 50	OK
3965......	89, 93	18.8	23.0	3 40	7 38	5 15	8 32	"
4035......	96, 97	17.5	23.0	3 45	9 00	6 00	9 30	"

The sieves used in all of these tests were what are known as the Tyler standard sieves, manufactured by The W. S. Tyler Company, Cleveland, Ohio. The dimensions of the sieves used are given in Table 4. Table 5 gives the grading and other properties of the aggregates. The sieve analyses are the average from samples taken at intervals throughout the period covered by the tests.

The weight per cubic foot of the aggregate was determined by means of machined cast-iron measures, having capacities of ⅕ and ½ cu. ft. The ½ cu. ft. measure was used for the larger sizes; the ⅕, for the smaller sizes. The inside diameter of each measure is equal to the depth. The test was made by filling the measure about ⅓ full and puddling with a small steel bar pointed at the lower end. Filling and puddling was continued in like manner till the measure was full. After striking off with a straight-edge, the weight was determined.

TABLE 3.—STRENGTH TESTS OF CEMENT.

1 : 3 Standard Sand Mortar.
Values for strength tests are expressed in pounds per square inch.
Each value is the average of five tests made on different days.

Cement Lot No.	Water Used, per cent.	Tensile Strength. (briquettes)			Compressive Strength (2 by 4-in. cylinders).		
		7 days.	28 days.	3 months.	7 days.	28 days.	3 months.
3768	10.5	292	368	419	2120	3580	4320
3965	10.3	242	378	462	1770	3360	4380
4035	10.3	246	386	402	1700	2640	3750

TABLE 4.—SIZES OF SIEVES USED IN SIEVE ANALYSIS OF AGGREGATES.

Brass wire cloth sieves with square openings, manufactured by The W. S. Tyler Co., Cleveland, O.

Sieve Number or Size.	Size of Clear Opening.		Diameter of Wire, in.
	in.	mm.	
100	0.0058	0.147	0.0042
48	0.0116	0.295	0.0092
28	0.023	0.59	0.0125
14	0.046	1.18	0.025
8	0.093	2.36	0.032
4	0.185	4.70	0.065
⅜	0.37	9.40	0.092
¾	0.74	18.9	0.135
1½	1.5	38.0	.....

The density of the aggregate was computed from the specific gravity and the unit weight. The density refers to the total volume of solids in the mass. The voids in the aggregate is the complement of the density.

MIXING CONCRETE.

All machine-mixed concrete included in this report was mixed in a 3½-cu. ft. tilting concrete mixer manufactured by The T. L. Smith Company, of Milwaukee. This machine is known by the trade name of

"Mascot" mixer. The studies of the effect of time of mixing covered periods of ¼ to 10 minutes. The cement, water and aggregate for the batch were all weighed or measured and placed in the mixing drum before the mixer was started. The mixing period was measured from the time the mixer was started to the beginning of discharge.

In Series 93 a parallel set of compression cylinders was made, using hand-mixed concrete made in the manner regularly followed in concrete tests made in this Laboratory. In these tests a batch of concrete was proportioned of the size required for one cylinder (about ⅕ cu. ft.). Each batch was mixed by hand with a trowel in a shallow metal pan. The methods of moulding, etc., were the same for all specimens. These tests were carried out for the purpose of securing a direct comparison between the strength of hand-mixed and machine-mixed concrete.

TABLE 5.—NOTES ON AGGREGATES.

All aggregates used in these tests consisted of sand and pebbles from the Chicago Gravel Co.'s plant near Elgin, Ill.

Reference No. in Series.	Range in Size.	Amount Finer than Each Sieve, Per Cent by Weight.										Weight, lb. per cu. ft.	Density.	Voids.
		100	48	28	14	8	4	⅜	¾	1¼	2			
SERIES 81.														
All............	0–1¼ in.	0	2	11	18	28	40	50	80	100	...	130	0.781	0.219
SERIES 89.														
All............	0–1¼ in.	0	3	11	16	21	26	44	82	100	...	130	0.781	0.219
SERIES 93.														
49–54, 249.....	0–14	2	11	58	100	...	...	...	...	...	...	104	0.626	0.376
55–60, 255.....	0– 8	1	8	42	72	100	...	...	...	...	...	110	0.660	0.340
61–66, 261.....	0– 4	1	6	30	52	72	100	...	...	...	...	115	0.691	0.309
67–72, 267.....	0–⅜ in.	1	5	23	40	56	77	100	...	...	...	124	0.746	0.254
73–78, 273.....	0–¾ in.	0	4	21	25	35	49	69	100	...	...	126	0.756	0.244
1–48, 201, 243	0–1¼ in.	0	2	10	18	24	34	51	84	100	...	130	0.781	0.219
79–84, 279.....	0–2 in.	0	1	6	10	14	20	40	60	80	100	128	0.770	0.230
SERIES 96.														
All............	0–1¼ in.	0	2	10	18	24	34	51	84	100	...	130	0.781	0.219
SERIES 97.														
All............	0–1¼ in.	0	2	10	18	24	34	51	84	100	...	130	0.781	0.219

The different rates of rotation of the mixer drum used in Series 96 were secured by shifting the driving belt to pulleys of different size or by varying the speed of the motor.

The water tank which supplied the concrete mixer was calibrated in such a way that accurate readings of the quantity of water could be secured. Both hot and cold water were available for mixing purposes. The variations in temperature of mixing water in Series 97 were secured by this means. The extremely low temperatures were reached by placing ice in the water tank. The temperatures near the boiling point were obtained by heating the water tank by means of gas jets.

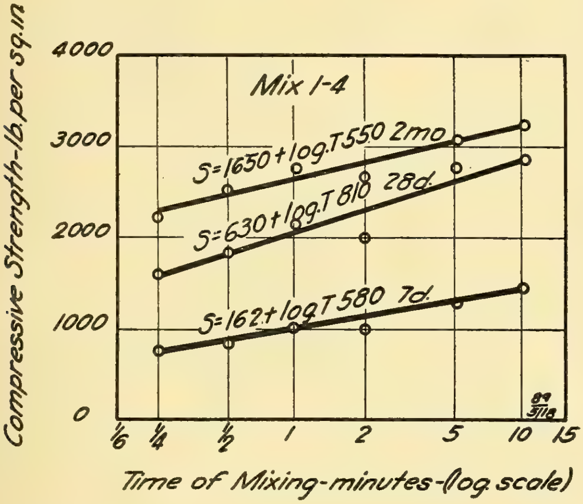

FIG. 5.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 89—Same data as in Fig. 4, except time of mixing is platted to logarithmic scale. The equations require that time be expressed in seconds. Compare Fig. 6, 22, 24, 26, 28 and 30.

Fig. 1 shows the mixer used in these tests. The water tank can be seen in the background. In the foreground is shown a trough which was used for receiving the batch as it was discharged from the mixer. The trough is about 5 by 14 in. in section, 15 ft. long; the bottom is lined with galvanized sheet steel; partitions of ⅛-in. steel plates are provided at intervals of about 15 in. The sides are sloped slightly to facilitate placing the partitions. It is mounted on castors in such a way that it can be pulled over the floor so as to distribute the concrete throughout its length as the mixer is discharged. The partitions prevent the concrete from flowing from one point to another in the batches mixed to very wet consistencies.

TEST PIECES.

Most of the tests covered in this report consist of compression tests of 6 by 12-in. concrete cylinders. The wear tests in Series 89 were made on concrete blocks 8 in. square and 5 in. in thickness. The hand-mixed specimens in Series 93 were each mixed and proportioned separately. All proportions are expressed in terms of the volume of cement and mixed aggregate. For instance a 1–4 mix would be made up of 1 cu. ft. of cement (equivalent to 1 sack) and 4 cu. ft. of aggregate, without regard to the size or grading of the aggregate. A 1–4 mix was used in most of the tests; this corresponds to the usual 1–2–3 or 1–1½–3 mix. The exact equivalent of the 1–4 mix expressed in the customary manner would vary with the size and grading of the aggregates.

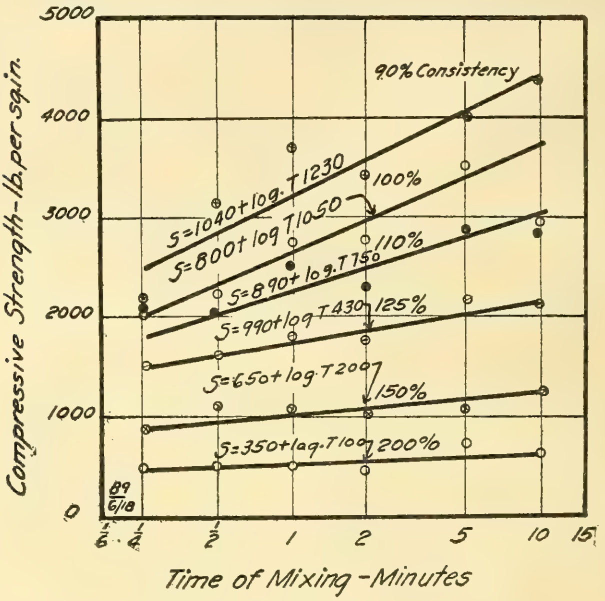

FIG. 6.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 89—28-day tests. Each value is the average of 24 tests. Similar curves may be drawn from the data for tests made at 7 days and 2 months. The equations require that time of mixing be expressed in seconds.

Compare Fig. 22, 24, 26, 28, 30.

The consistency of concrete was varied in most of the series. In the tables and diagrams we have used the term "relative consistency." What we have called "100 per cent. consistency" is a plastic mix which when molded in the form of a 6 by 12-in. cylinder will show a shortening of about ¼ to ½ in. if the steel form is slipped off immediately after molding. The quantity of water required to produce this condition varies of course with the proportions of the mix, the grading and absorption of the aggregate, etc. Other consistencies for the same mix are expressed in terms of the quantity of water required to produce "normal consistency," *e. g.*, 90 per cent. or 150 per cent. It should not be assumed that this "normal consistency" gives con-

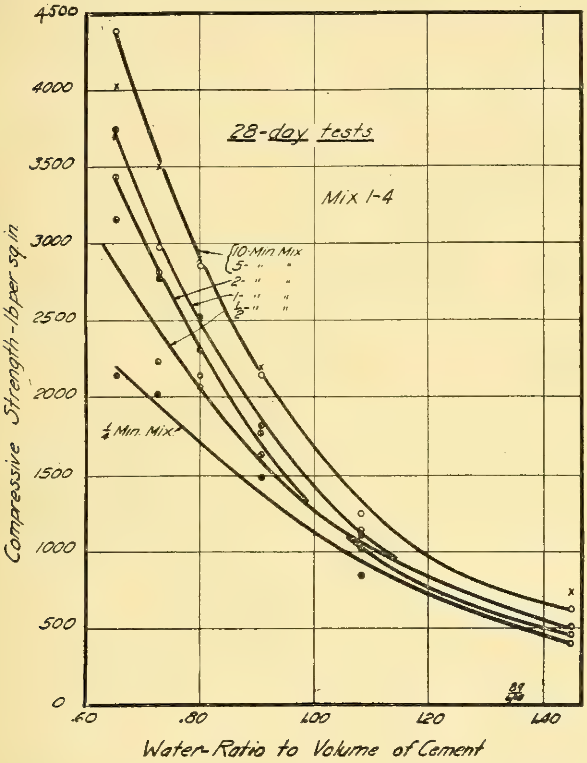

FIG. 7.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 89—Water-Strength curves for each time of mixing. Each value is the average of 4 tests from the same batch. Water content calculated as a ratio of the volume of cement. Similar curves may be drawn from data for tests made at 7 days and 2 months. Compare Fig. 9, 31, 33, 44, 52.

crete of the highest strength. Highest strength has generally been found at consistencies around 90 per cent. of normal.

While the method of expressing the consistency by reference to a certain "normal" is most convenient, it should be stated that the method of testing for this consistency described above is not entirely satisfactory. For lean or very rich mixes or for aggregates of unusual fineness or coarseness the test does not give concordant results. A penetration test for consistency was used in Series 97. In this test a bar ¾ in. in diameter, 24 in. long, weighing 800 g. was used. The bar was made of thin brass tubing with a pointed steel shoe at one end. The steel point was tapered from $\frac{1}{16}$ in. to ¾ in. in a length of 2 in. It was held in a vertical position over the top of the cylinder with the pointed end in contact with the fresh concrete. Upon being released the bar, falling of its own weight, penetrated the concrete a certain distance, depending on the viscosity of the mass. This test was repeated several times, the depth of penetration in inches measured and the average recorded. If the point struck a large piece of aggregate and did not penetrate, that trial was not considered.

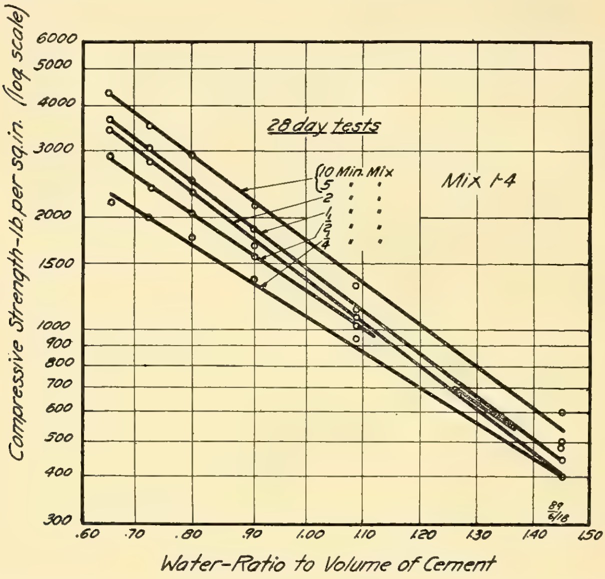

FIG. 8.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 89—Same data as in Fig. 7, except strength platted to logarithmic scale. The points are platted from the curves in Fig. 7.

Compare Fig. 10, 32, 34, 45, 53.

This method gave fair results in Series 97 (see Fig. 47); however, it should be borne in mind that only one mix was used and there was no variation in the character or grading of the aggregate. The method is not satisfactory for wide ranges of mixes or grading of aggregates, but appears to be promising for use where only a limited range in the character of the concrete will be encountered.

The amount of water used in the batch exerts such a controlling influence that the quality of the concrete depends very largely on this factor. The quantity of water to be used depends on the following factors:

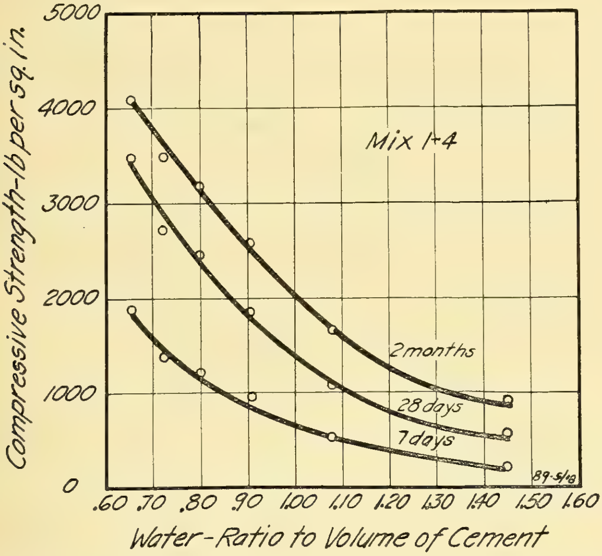

FIG. 9.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.
Series 89—Each point is the average of 24 tests, from 6 times of mixing.
Compare Fig. 7, 31, 33, 44, 52.

1. The quantity of cement (mix).
2. The normal consistency of the cement.
3. The size and grading of the aggregate.
4. The absorption of the aggregate.
5. The moisture already present in the aggregate.
6. The nature of the work, that is, the size of the members, the methods of handling, placing, finishing, etc.

The term "consistency" as used in this report refers to the physical condition of the fresh concrete with reference to viscosity or plasticity and not to the quantity of water. It has been found convenient to express the quantity of water used in the concrete in terms of the volume of the cement. This ratio has been found to be the best criterion of the strength of the concrete. The whole question of consistency has been studied, but it is not feasible to take up a detailed discussion of these studies in the present report.

The 6 by 12-in. compression cylinders were molded in metal forms made of 12-in. lengths of 6-in. inside diameter cold drawn steel tubing which had been split along one element by means of a thin slotter. The form was closed by a circumferential band. Each form stood on a machined cast-iron base plate. A sheet or paraffined tissue paper was placed between the base plate and the cylinder form.

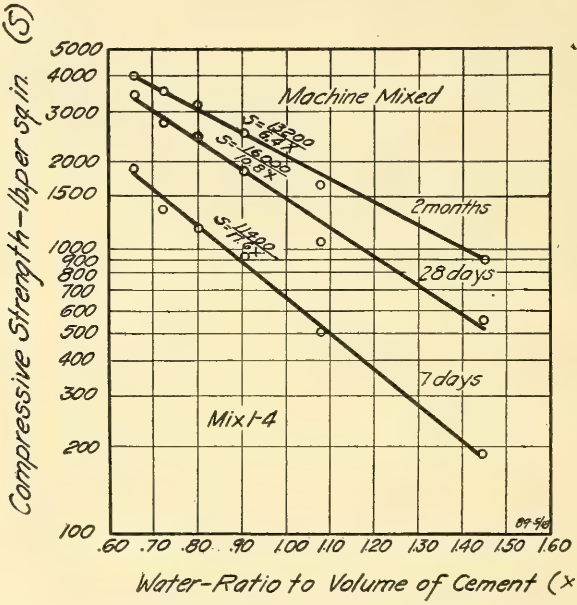

FIG. 10.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 89—Same data as in Fig. 9, strength platted to logarithmic scale. The points are platted from the curves in Fig. 9. Each value is the average of 24 tests, from 6 times of mixing.

Compare Fig. 8, 10, 32, 34, 45, 53.

The concrete for the machine-mixed batches was taken from the receiving trough by means of a large hand scoop. In molding the cylinder the form was filled about one-third full and the concrete puddled with a ¾-in. steel bar about 21 in. long. This process was continued until the form was filled. The top was leveled off with a bricklayer's trowel. About 3 to 6 hours after molding a thin layer of neat cement paste (which was mixed at the same time or before the concrete) was spread over the top of the cylinder. A piece of plate glass and a sheet of paraffined paper was used to form a cap which made a smooth, square end for loading. The glass remained in place until the form was removed. This method of capping is much better than setting the specimens in plaster of paris or cement-plaster caps immediately before testing. It has the following advantages:

TABLE 6.—UNIFORMITY OF MACHINE-MIXED CONCRETE (SERIES 81).

Compression tests of 6 by 12-in. cylinders.
Smith "Mascot" tilting mixer—capacity 3½ cu. ft.
1 : 4 mix by volume.
Aggregates—sand and pebbles; graded 0–1¼ in.
Time of mixing, 1 minute.
Specimens stored in damp sand.
The cylinders in a batch were numbered consecutively; the odd numbers tested at 7 days, the even numbers at 28 days.

Reference No.	Water.		Compressive Strength.				Weight, lb. per sq. ft.	Density.	Yield.
	Relative Consistency.	Ratio to Cement by Volume.	lb. per sq. in.		Mean Error,* per cent.				
			7 days.	28 days	7 days.	28 days.			
A.........	100	0.78	1340	2640			147.5	0.804	1.121
			1330	2520					
			1310	2500					
			1320	2420					
			1390	2470					
			1380	2440					
			1340	2340					
			1340	2480	2.3	3.8			
B.........	115	0.89	990	1920			146.5	0.790	1.142
			990	1990					
			1000	2020					
			1010	2010					
			960	2050					
			1010	2030					
			990	1860					
			....	2120					
			990	2000	1.8	4.0			
C.........	130	1.00	870	1580			146.0	0.778	1.160
			950	1890					
			920	2020					
			1000	1810					
			970	2110					
			890	2160					
			1080	2200					
			....	1920					
			950	1960	7.5	10.5			

* Computed from formula $E = \frac{100}{A}\sqrt{\frac{\Sigma v^2}{n-1}}$ Where v is the difference of any value from average,
n is the number of values,
A is the average.

1. The cap is just as strong and stiff as the concrete, and forms an integral part of the specimen.
2. The time and labor required is a small part of that necessary with the plaster method.

3. The plate glass prevents evaporation of water during the period the concrete is in the form.

4. The cylinder is ready for test at any time without further preparation.

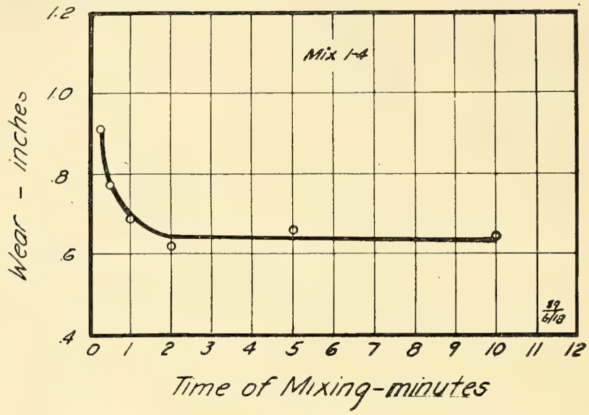

FIG. 11.—EFFECT OF TIME OF MIXING ON THE WEAR OF CONCRETE.
Series 89—Age 2 mo. Each value is the average of 30 tests, 5 each, from 6 different consistencies.

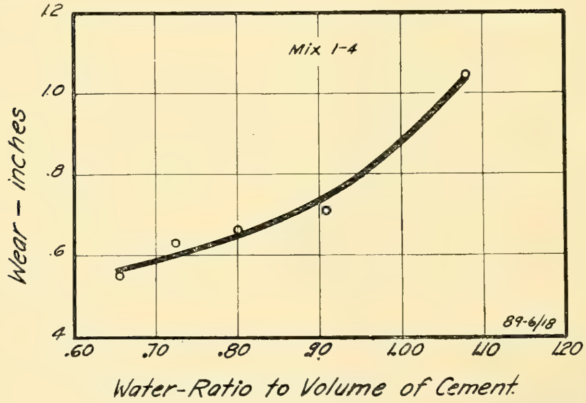

FIG. 12.—INFLUENCE OF WATER ON THE WEAR OF CONCRETE.
Series 89—Age 2 mo. Each point is the average of 30 tests, 5 each, from 6 different times of mixing.

The forms for the wear blocks were made in gangs of 3. The forms were set on a sheet of building paper laid directly on the concrete floor. Each form was filled before puddling. The top was leveled off with a trowel. After a period of 1 to 2 hours the tops of

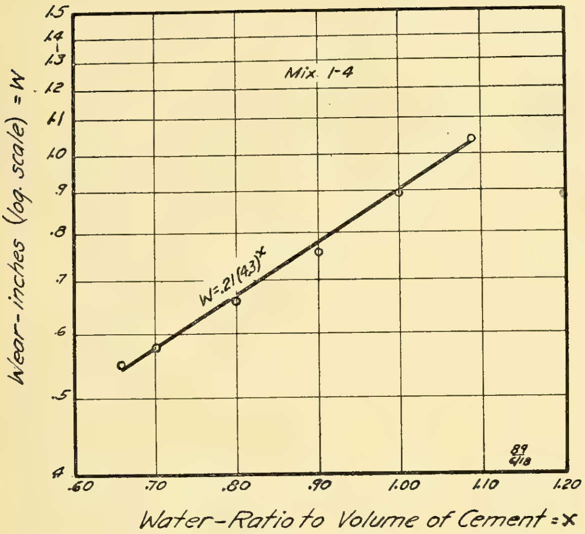

FIG. 13.—INFLUENCE OF WATER ON THE WEAR OF CONCRETE.
Series 89—Same data as in Fig. 12, except depth of wear is platted to logarithmic scale.

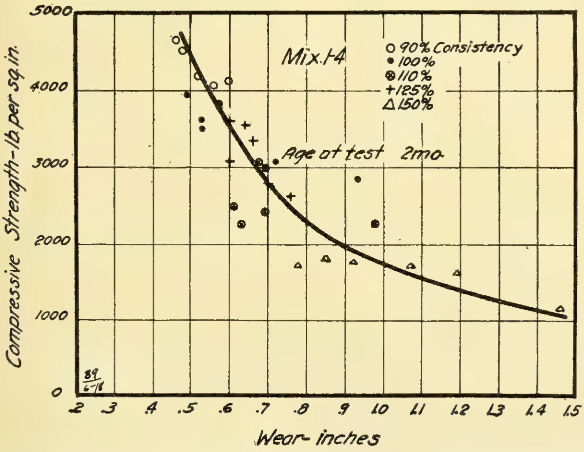

FIG. 14.—RELATION BETWEEN STRENGTH AND WEAR OF CONCRETE.
Series 89—Each point is based on 4 compression tests and 5 wear tests from the same batch. Compare Fig. 16 from a series of tests on hand-mixed concrete.

the blocks were finished by hand with a wood float. Instead of capping, the blocks were covered with a sheet of wet building paper and about 3 in. of damp sand. This method prevented loss of water while the blocks were in the forms.

All test pieces were allowed to remain in the metal forms over night. Upon removal of the forms they were stored in damp sand in the basement. The compression cylinders were not removed from the damp sand until immediately preceding their test. The wear blocks, however, were allowed to dry out in an open room for two days prior to testing.

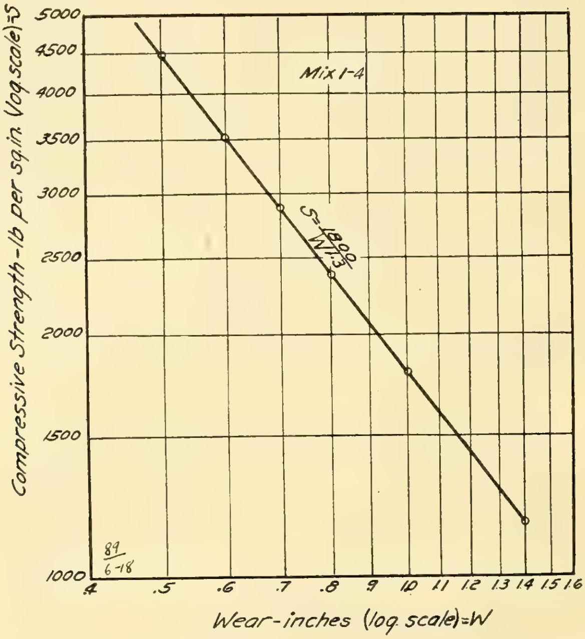

FIG. 15.—RELATION BETWEEN STRENGTH AND WEAR OF CONCRETE.

Series 89—Same data as in Fig. 14; both strength and depth of wear platted to logarithmic scale. Compare Fig. 17.

The machine-mixed batches consisted of 15 to 17 specimens. The specimens were made and numbered in the same order in all cases. The material which was first discharged from the mixer was used to make Specimen 1; the material last discharged formed the last specimen from the batch. In assigning the test pieces to be tested at each age, the specimens were distributed throughout the batch. The exact distribution is stated in the tables giving the results of the tests in each series.

METHODS OF TESTING.

The compression tests of concrete were made in a 200,000-lb. Olsen universal testing machine. A spherical bearing block was used on top of the specimen. Stress-deformation measurements were made from which the modulus of elasticity of the concrete can be computed. These values are omitted from the present report.

Wear tests of concrete were made in the Talbot-Jones rattler. The test pieces consist of blocks 8 in. square and 5 in. in thickness.

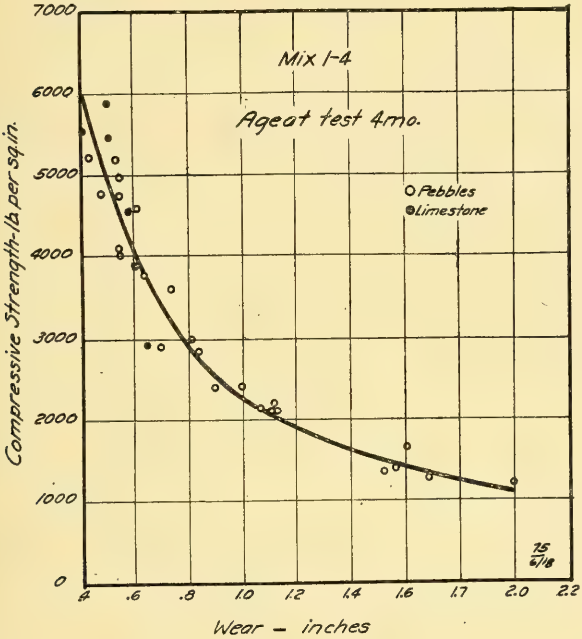

FIG. 16.—RELATION BETWEEN STRENGTH AND WEAR OF CONCRETE.

Series 75—Data similar to Fig. 14; 1:4 hand-mixed concrete; age 4 mo. Each value is based on 5 compression tests and 5 wear tests. Detailed data from Series 75 (Effect of Consistency and Curing Condition on the Wear of Concrete) not given in this report.

The blocks are arranged around the perimeter of the drum of the rattler as shown in Fig. 2. Ten blocks constitute a test set. The ten-side polygon formed by the test blocks presents a nearly continuous surface. The outside diameter of the polygon thus formed is 30 in. and the inside diameter 26 in. During the test the front of the chamber is closed by means of a light steel plate. The abrasive charge consists of 200 lb. of cast-iron balls (about 133 1⅞ in. and 10 3¾ in. in diameter). These balls conform to the requirements of the standard rattler test of paving brick of the American Society for Testing Materials.

The test consists of exposing the inner faces of the concrete blocks to the wearing action of the charge for 1800 revolutions at the rate of 30 r. p. m. The machine was run for 900 revloutions in one direction and then reversed. Two sets of blocks are tested at once in the machine now in use. Each block was weighed upon removal from the form, upon removal from damp sand, immediately before and after testing. The loss in weight during the test is used as a measure of the wear. This loss is reduced to an equivalent depth of wear in inches.

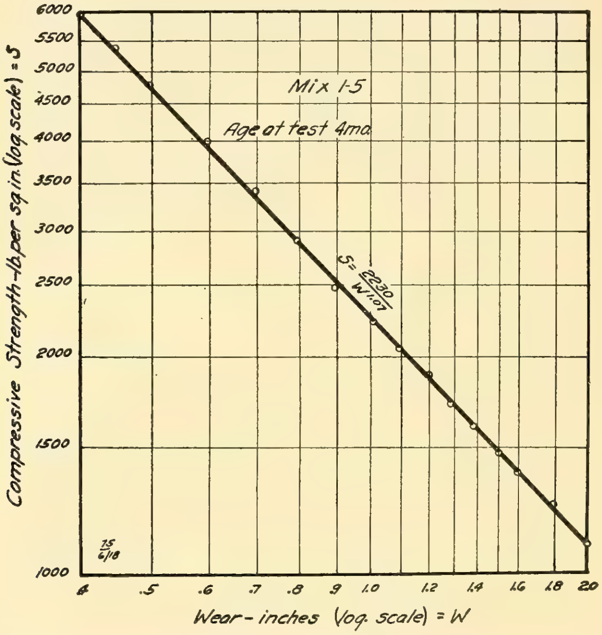

FIG. 17.—RELATION BETWEEN STRENGTH AND WEAR OF CONCRETE. Same data as in Fig. 16; both strength and depth of wear platted to logarithmic scale. Compare Fig. 15.

This method of making wear tests of concrete is believed to have the following advantages as compared with other methods which have been used or proposed for this purpose:

1. The concrete is subjected to a treatment which approximates that of service.

2. The test piece is of usual form and of sufficient size that representative concrete can be obtained.

TABLE 7.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 89).

(EFFECT OF CONSISTENCY.)

Mix 1 : 4 by volume.
Smith tilting "Mascot" mixer; capacity 3½ cu. ft.
Compression tests made on 6 by 12-in. cylinders.
Wear tests made in Talbot-Jones rattler; specimens consisted of concrete blocks 8 by 8 by 5 in. thick; 10 wear blocks in a ring were run for 1800 revolutions at 30 r.p.m., with a charge of 200 lb. cast iron shot.
Portland cement—equal parts of 4 brands purchased in Chicago market.
Aggregates—sand and pebbles from Elgin, Ill.; graded (0–1¼ in.) same for all batches.
From each batch 12 cylinders and 5 wear blocks were tested at ages shown below, the specimens being numbered in the order of molding.

Kind of Specimen.	Specimens Tested at a Given Age.		
	7 days.	28 days.	2 months.
Compression cylinders	2, 6, 10, 14	3, 7, 11, 15	4, 8, 12, 16
Wear blocks	....	....	1, 5, 8, 13, 17

Cylinders stored in damp sand; tested damp. Wear blocks stored in damp sand until 2 days prior to test.

Reference No.	Time of Mixing, min.	Tests of 6 by 12-in. Cylinders.											Wear Tests at 2 Months.	
		Compressive Strength.							Weight, lb. per cu. ft.	Density.	Yield.		Depth of Wear, in.	Mean Error, per cent.
		lb. per sq. in.			Mean Error, per cent.									
		7 days	28 days	2 mo.	7 days	28 days	2 mo.	Av.						
RELATIVE CONSISTENCY 90 PER CENT. WATER, 0.66 OF VOLUME OF CEMENT.														
1	10	2510	4370	4660	1.9	2.0	4.1	2.7	158.0	0.871	1.036		0.46	10.2
2	5	2230	4020	4520	4.2	7.9	4.0	5.4	158.0	0.871	1.036		0.48	4.0
3	2	1740	3440	4060	1.2	9.9	15.3	8.8	157.5	0.868	1.040		0.56	6.0
4	1	1840	3750	4130	9.2	5.2	12.5	9.0	157.5	0.868	1.040		0.60	8.4
5	½	1620	3180	4190	8.0	8.6	7.4	8.0	156.0	0.860	1.049		0.52	7.9
6	¼	1260	2140	2880	31.4	32.0	20.5	28.0	153.0	0.844	1.070		0.69	19.9
Aver.		1870	3480	4070	9.3	10.9	10.6	10.3	157.0	0.864	1.045		0.55	9.4
RELATIVE CONSISTENCY, 100 PER CENT. WATER, 0.73 OF VOLUME OF CEMENT.														
7	10	1640	2930	3960	4.0	3.5	7.5	5.0	156.0	0.856	1.055		0.49	16.9
8	5	1750	3520	3810	3.2	4.0	20.4	9.2	156.5	0.857	1.052		0.57	9.0
9	2	1460	2800	3500	5.0	4.6	8.2	5.9	156.0	0.856	1.055		0.53	5.7
10	1	1380	2780	3620	5.7	4.2	9.8	6.6	156.0	0.856	1.055		0.53	5.7
11	½	1130	2210	3080	5.7	7.6	6.7	6.7	155.0	0.851	1.061		0.72	8.3
12	¼	1000	2020	2860	22.6	19.3	9.2	17.0	153.0	0.839	1.077		0.93	20.2
Aver.		1390	2710	3470	7.7	7.2	10.3	8.4	155.4	0.852	1.059		0.63	11.0
RELATIVE CONSISTENCY, 110 PER CENT. WATER, 0.80 OF VOLUME OF CEMENT.														
13	10	1520	2860	3640	12.8	12.5	8.5	11.3	156.5	0.852	1.062		0.60	7.2
14	5	1500	2900	3550	6.4	3.6	7.8	5.9	156.5	0.852	1.062		0.64	7.4
15	2	1090	2300	3070	14.2	11.4	7.2	10.9	155.5	0.845	1.068		0.60	5.4
16	1	1160	2520	3350	4.9	8.8	5.9	6.5	155.5	0.845	1.068		0.68	8.9
17	½	880	2060	2620	24.1	16.2	13.0	17.8	154.5	0.842	1.074		0.76	25.5
18	¼	1060	2140	2790	21.4	28.0	22.6	24.0	153.0	0.834	1.086		0.70	16.0
Aver.		1200	2460	3170	14.0	13.4	10.8	12.7	155.2	0.845	1.070		0.66	11.7

TABLE 7.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 89).—*Continued.*

Reference No.	Time of Mixing, min.	Tests of 6 by 12-in. Cylinders.										Wear Tests at 2 Months.	
		Compressive Strength.							Weight, lb. per cu. ft.	Density.	Yield.	Depth of Wear, in.	Mean Error, per cent.
		lb. per sq. in.			Mean Error, per cent.								
		7 days	28 days	2 mo.	7 days	28 days	2 mo.	Av.					
RELATIVE CONSISTENCY, 125 PER CENT. WATER, 0.91 OF VOLUME OF CEMENT.													
19	10	1260	2140	2980	0.9	14.5	13.1	9.5	154.5	0.833	1.084	0.69	11.6
20	5	1200	2170	3030	7.6	15.5	4.6	9.2	155.0	0.835	1.081	0.68	11.5
21	2	1030	1800	2500	14.1	7.6	12.0	11.3	153.5	0.827	1.091	0.61	13.0
22	1	900	1830	2250	9.9	8.6	16.8	11.8	153.5	0.827	1.091	0.63	4.5
23	½	710	1640	2400	17.8	22.9	5.8	15.5	154.0	0.830	1.088	0.69	8.7
24	¼	690	1510	2250	25.0	44.6	42.5	37.4	153.0	0.825	1.095	0.98	27.6
Aver.		960	1850	2570	12.6	19.0	15.8	15.8	153.9	0.830	1.088	0.71	12.8
RELATIVE CONSISTENCY, 150 PER CENT. WATER, 1.08 OF VOLUME OF CEMENT.													
25	10	690	1240	1820	5.4	9.0	7.4	7.3	153.0	0.810	1.115	0.85	9.4
26	5	560	1090	1780	12.1	21.6	5.6	13.1	152.5	0.808	1.118	0.92	12.7
27	2	480	1030	1750	9.3	8.5	6.1	8.0	153.5	0.814	1.110	0.78	20.5
28	1	500	1090	1740	12.1	9.2	8.8	10.0	152.5	0.808	1.118	1.06	17.4
29	½	500	1120	1690	4.2	17.2	12.9	11.4	152.0	0.806	1.120	1.19	8.4
30	¼	340	880	1200	29.0	36.2	21.7	29.0	151.5	0.803	1.125	1.46	2.4
Aver.		510	1080	1660	12.0	17.0	10.4	13.1	152.5	0.808	1.118	1.04	11.8
RELATIVE CONSISTENCY, 200 PER CENT. WATER, 1.45 OF VOLUME OF CEMENT.													
31	10	260	620	1110	26.9	31.8	28.4	29.0	149.0	0.764	1.181	....	....
32	5	260	730	1040	18.0	25.4	28.2	23.9	150.0	0.768	1.176	....	....
33	2	160	460	720	10.8	15.6	12.1	12.8	148.0	0.759	1.190	....	....
34	1	150	510	880	11.5	6.7	5.8	8.0	149.0	0.764	1.181	....	....
35	½	160	500	940	14.0	15.7	4.5	11.4	148.0	0.759	1.190	....	....
36	¼	150	460	840	36.0	42.4	29.0	35.8	147.0	0.752	1.200	....	....
Aver.		190	560	920	19.5	22.9	18.0	20.1	148.5	0.761	1.186		
GRAND AVERAGE FOR ALL CONSISTENCIES.													
	10	1450	2570	3210	7.8	10.9	10.9	....	155.0	0.834	1.085	0.64*	10.1*
	5	1280	2470	3040	8.3	11.7	11.2	....	154.0	0.835	1.084	0.66*	8.7*
	2	1010	2020	2670	9.8	9.9	9.7	....	154.0	0.831	1.089	0.62*	9.3*
	1	1010	2140	2760	8.3	7.4	9.4	....	154.0	0.830	1.089	0.69*	9.0*
	½	840	1820	2500	14.0	14.9	9.1	....	153.0	0.827	1.094	0.77*	14.0*
	¼	790	1610	2230	26.7	32.9	27.0	....	152.0	0.819	1.106	0.91*	17.0*

* Does not include 200 per cent consistency.

3. The test pieces are convenient to make, store and handle, and require a relatively small amount of concrete.

4. The cost of tests is not excessive.

5. The machine used is found in numerous testing laboratories.

6. The wearing action takes place on the top or finished surface of the concrete. This makes it possible to study the effect of various surface treatments or finishes.

7. Several tests may be made at the same time, thus enabling more representative results to be obtained.

8. Tests may be made on sections of concrete cut from roads which have been in service.

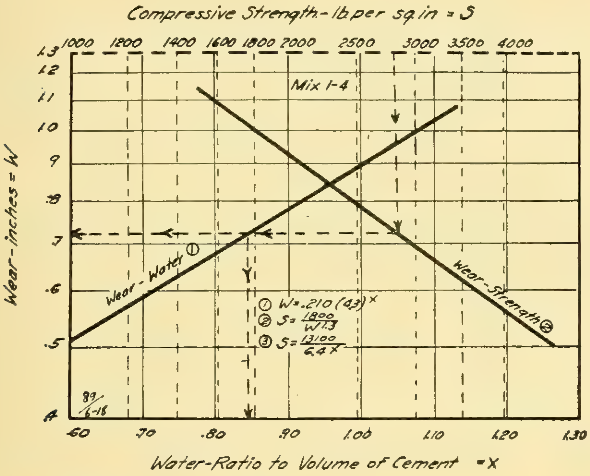

FIG. 18.—INFLUENCE OF WATER ON THE STRENGTH AND WEAR OF CONCRETE.

Series 89—A combination of Fig. 13 and 15. The strength-water relation may be platted on the same diagram if desired; equation (3) gives the relation for these tests.

9. Other paving materials such as brick, granite blocks, etc., may be tested in the same way as the concrete.

The foregoing notes are substantially the same as given in a paper by the writer on "A Method of Making Wear Tests of Concrete," Proceedings, American Society for Testing Materials, Part II, p. 194, 1916.

A comprehensive series of tests on the effect of size and grading of aggregates, using gravel, crushed limestone, granite and slag, is now under way. Tests have also been made on a large number of wear blocks molded in the field on concrete road construction.

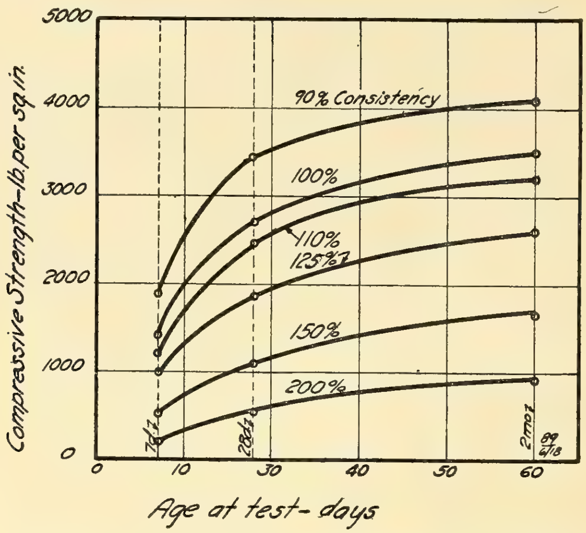

FIG. 19.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.
Series 89—Each point is based on 24 tests obtained from averaging 6 times of mixing. Compare Fig. 21 and 35.

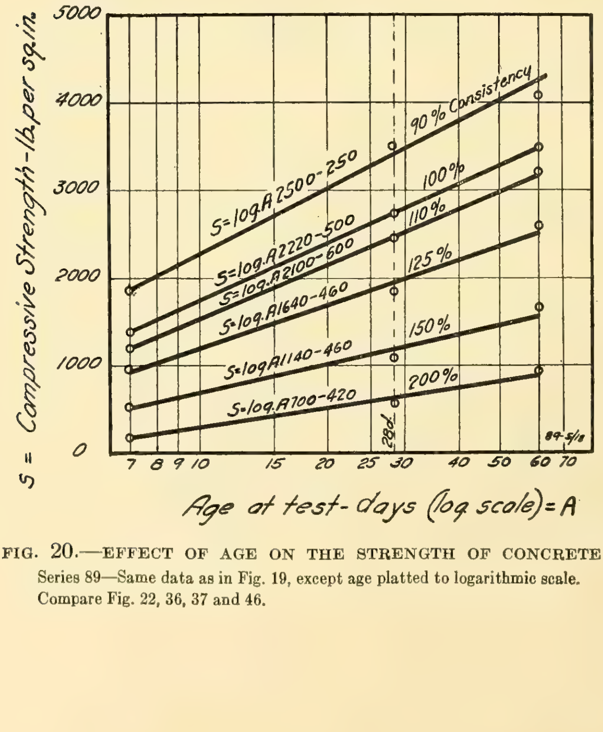

FIG. 20.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.
Series 89—Same data as in Fig. 19, except age platted to logarithmic scale. Compare Fig. 22, 36, 37 and 46.

DATA OF TESTS.

The data of the tests are given in Tables 6 to 10 inclusive. Notes accompanying the tables give details of the mixes, etc., for each series. Only average values are given for compressive strength and wear, except in Series 81, where the results of individual tests are given. Values for compressive strength are the average of four or five tests from the same batch. In Series 97, "Effect of Temperature of Mixing Water," duplicate batches were mixed on different days. In Series 96, "Effect of Rate of Rotation of Mixer Drum," triplicate batches were mixed on different days for one consistency, for the purpose of studying the uniformity of such tests.

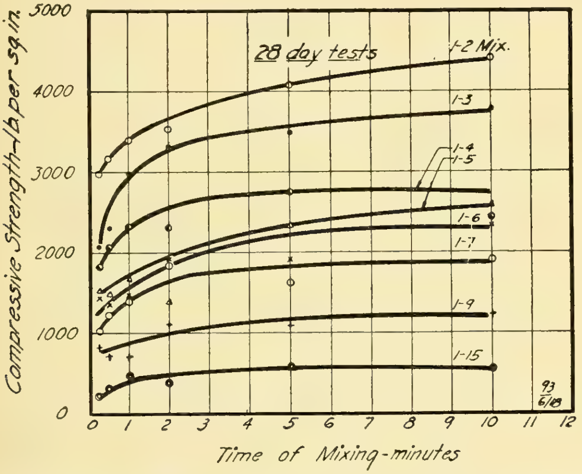

FIG. 21.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 93—All aggregates graded 0–1¼ in. Each value is the average of 4 tests. Similar curves may be drawn for tests at 7 days. For similar tests at 3 months see Fig. 23.

Compare Fig. 4, 23, 25, 27, 29.

Most of the data from the tests have been platted in the diagrams in Fig. 4 to 53 inclusive. In several instances curves have been platted for only a portion of the values; for instance, more emphasis has been given to the 28-day than to the 7-day tests.

Owing to numerous instances of overlapping in the different test series it seems best to discuss the data under several headings, instead of taking up each series separately.

EFFECT OF CONSISTENCY ON THE STRENGTH OF CONCRETE.

The remarkable influence of the quantity of mixing water on the strength and other properties of the concrete has been demonstrated very forcefully in all series in which variations in consistency were introduced. Owing to the importance of this factor in any consideration of the tests the discussion of the effect of consistency is given first.

Many different methods have been used for indicating the consistency of concrete: (1) Total water as a percentage of the weight of the dry materials; (2) total water as a percentage of the weight of the finished concrete; (3) total water as a percentage of the weight of the cement. The first method is the one which is universally used in cement and mortar tests. In the tests made in this Laboratory we have preferred to express the water in terms of the volume of the cement, considering 94 lb. of cement as 1 cu. ft. This method of indicating the water content of concrete mixes is not essentially different from (3). However, since volumetric measurements are used for cement and aggregate it seems desirable to use volumes for the water also, for sake of uniformity. Methods (1) and (2) above are objectionable, since with the same quantity of water, widely different values are obtained, due to variation in the unit weights of the aggregate.

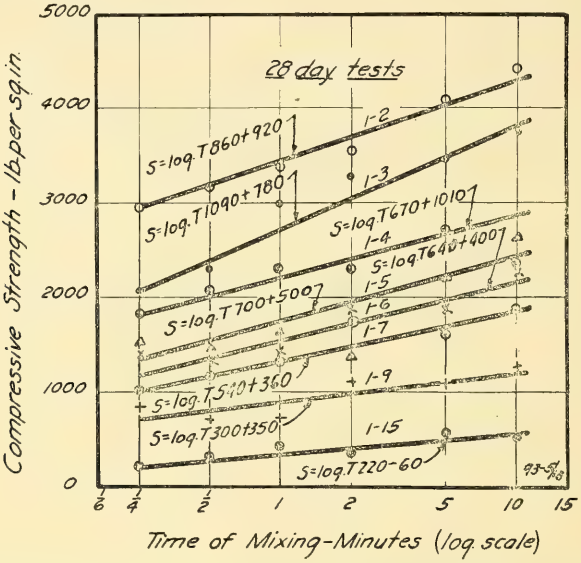

FIG. 22.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 21, except time of mixing is platted to logarithmic scale.
The equations require that time of mixing be expressed in seconds.
Compare Fig. 5, 6, 24, 26, 28, 30.

Under "Test Pieces," above, a list was given of the factors which determine the quantity of water which must be placed in the batch. It should be borne in mind that all of the water used should not be credited to the cement; a certain quantity will be absorbed by the aggregate. In some instances the absorbed water is a considerable proportion of the whole amount. The water absorbed by the aggregate should be deducted in computing the volume of water which affects the action of the cement.

For given concrete materials (that is, cement, water and aggregate) the amount of water which affects the cement may be varied in the following ways:

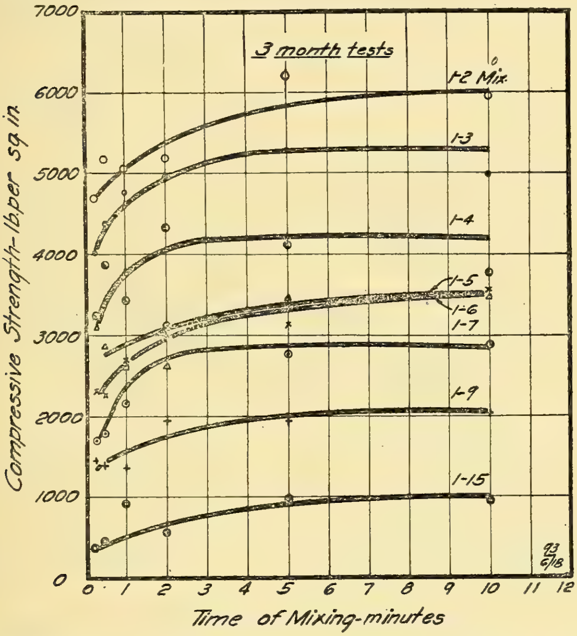

FIG. 23.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 93—3-mo. tests, similar to Fig. 21. A set of curves of the same kind may be platted for the 7-day tests.

Compare Fig. 4, 21, 25, 27, 29.

1. For the same mix and same grading of aggregate use different quantities of water, thus producing concretes of varying degrees of "wetness."
2. Change the quantity of cement (the mix) with the same grading of aggregate, mixing to the same plastic condition. The richer mixes will, of course, require a smaller percentage of water as compared to the cement than the lean mixes.

3. With the same mix and the same relative plasticity, change the grading of the aggregate from coarse to fine, etc. The mixes with a preponderance of fine material will require most water.

4. Any combination of two or all of the above methods may be used simultaneously; for instance, we may wish to compare a stiff 1–2 mix, using sand graded 0–28 with a wet 1–5 mix, using aggregate graded 0—1½ in.

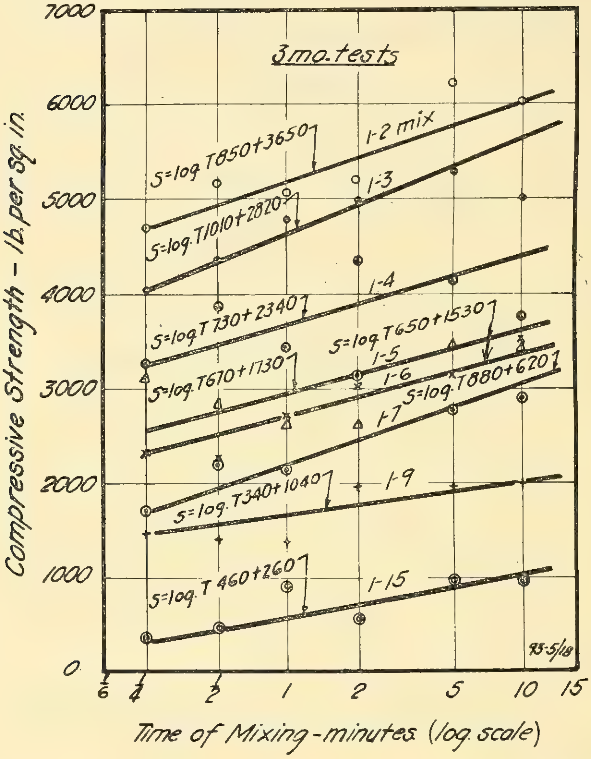

FIG. 24.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 23, except time of mixing platted to logarithmic scale.
The equations require that time of mixing be expressed in seconds.
Compare Fig. 5, 6, 22, 26, 28, 30.

In (4) every factor which affects the strength (the quantity of cement, grading of aggregate and relative consistency) has been changed. It will be shown below that we have in all cases a common basis for comparison of strength in the "water ratio" of the batch;

TABLE 8.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 93).

(EFFECT OF MIX AND SIZE OF AGGREGATE.)

Compression tests of 6 by 12-in. cylinders.
Mix by volume.
Portland cement—equal parts of 4 brands purchased in Chicago market.
All concrete in this series was mixed to 110 per cent relative consistency.
Aggregates—sand and pebbles from Elgin, Ill.
Smith tilting "Mascot" mixer; capacity 3½ cu. ft.
Mixing was begun after all materials were in the drum.

Machine-mixed cylinders made 16 in a batch; 4 for test at each age. In the order of making No. 1, 5, 9 and 13 were tested at 7 days; No. 2, 6, 10 and 14 at 28 days, etc.

Only one batch of machine-mixed concrete was made for each mix, grading and time of mixing. The mixing of the batches for a given mix and grading were distributed over a period of 4 weeks. All the 10-min. batches were mixed first, the 5-min. next, etc.

Hand-mixed specimens were made for comparison with machine-mixed concrete.

Specimens stored in damp sand; tested damp.

Reference No.	Mix by Volume.	Water.	Aggregate.			Time of Mixing, min.	Concrete.					
		Ratio to Cement by Volume.	Range in Size.	Weight, lb. per cu. ft.	Density.		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
							7 days	28 days	3 mo.			
MACHINE-MIXED CONCRETE.												
1	1:2	0.60	0–1¼	130.0	0.780	10	2440	4410	5960	153.4	0.803	1.276
2						5	2420	4080	6220	151.5	0.792	1.293
3						2	1760	3550	5180	152.8	0.799	1.281
4						1	1330	3400	5060	152.0	0.794	1.286
5						½	1420	3160	5160	150.0	0.784	1.305
6						¼	1180	2970	4690	150.2	0.786	1.303
Aver....							1760	3600	5380	151.6	0.793	1.291
7	1:3	0.70	0–1¼	130.0	0.780	10	2100	3760	4980	154.0	0.823	1.143
8						5	1720	3480	5260	153.2	0.820	1.148
9						2	1390	3300	4960	152.8	0.816	1.152
10						1	980	3000	4760	153.2	0.821	1.148
11						½	840	2300	4360	152.7	0.816	1.153
12						¼	810	2060	4000	151.2	0.809	1.164
Aver....							1310	2990	4720	152.8	0.818	1.151
13	1:4	0.81	0–1¼	130.0	0.780	10	1380	2380	3730	153.2	0.831	1.085
14						5	1340	2730	4100	153.2	0.831	1.085
15						2	1090	2310	4330	154.0	0.835	1.080
16						1	760	2320	3430	153.4	0.832	1.083
17						½	650	2060	3850	153.5	0.833	1.082
18						¼	600	1820	3240	152.2	0.826	1.092
Aver....							960	2270	3780	153.2	0.831	1.084
19	1:5	0.91	0–1¼	130.0	0.780	10	1380	2600	3460	154.1	0.830	1.057
20						5	1170	2340	3460	153.5	0.826	1.062
21						2	590	1370	2620	153.2	0.824	1.064
22						1	490	1670	2620	153.3	0.825	1.063
23						½	460	1490	2880	152.6	0.820	1.060
24						¼	460	1520	3150	153.0	0.823	1.065
Aver....							760	1830	3030	153.3	0.825	1.062

TABLE 8.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 93).—*Continued.*

Reference No.	Mix by Volume.	Water.	Aggregate.			Time of Mixing, min.	Concrete.					
		Ratio to Cement by Volume.	Range in Size.	Weight, lb. per cu. ft.	Density.		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
							7 days	28 days	3 mo.			
MACHINE-MIXED CONCRETE.—*Continued.*												
25	1:6	1.01	0–1¼	130.0	0.780	10	1260	2370	3510	154.4	0.849	1.013
26						5	860	1910	3150	153.2	0.844	1.020
27						2	760	1910	3040	153.5	0.845	1.018
28						1	440	1450	2640	153.1	0.843	1.021
29						½	450	1340	2270	153.3	0.845	1.019
30						¼	400	1440	2320	153.0	0.842	1.022
Aver.							700	1740	2820	153.4	0.845	1.019
31	1:7	1.10	0–1¼	130.0	0.780	10	1020	1920	2880	154.8	0.857	0.991
32						5	780	1640	2790	153.1	0.847	1.002
33						2	790	1870	3110	153.8	0.852	0.997
34						1	480	1420	2160	153.0	0.847	1.002
35						½	440	1240	2280	153.2	0.848	1.001
36						¼	350	1020	1700	152.8	0.845	1.004
Aver.							640	1520	2490	153.4	0.849	1.000
37	1:9	1.32	0–1¼	130.0	0.780	10	650	1240	2040	154.5	0.861	0.969
38						5	460	1100	1990	153.2	0.854	0.977
39						2	480	1110	1980	153.9	0.859	0.972
40						1	220	720	1370	151.3	0.844	0.989
41						½	280	720	1400	149.1	0.831	1.003
42						¼	260	810	1460	150.0	0.837	0.997
Aver.							390	950	1710	152.0	0.848	0.984
43	1:15	1.94	0–1¼	130.0	0.780	10	320	530	950	150.0	0.843	0.963
44						5	250	560	970	149.9	0.842	0.964
45						2	150	360	570	144.7	0.814	0.998
46						1	140	460	910	143.7	0.808	1.005
47						½	110	310	470	142.6	0.803	1.012
48						¼	110	240	390	141.8	0.797	1.018
Aver.							180	410	710	145.4	0.818	0.993
49	1:5	1.57	0–14	104.0	0.624	10	240	490	990	131.6	0.665	1.084
50						5	160	380	940	129.0	0.653	1.105
51						2	130	430	810	129.3	0.654	1.102
52						1	110	280	560	128.0	0.648	1.112
53						½	70	200	400	126.3	0.640	1.128
54						¼	20	*50	110	125.0	0.632	1.140
Aver.							120	310	640	128.2	0.649	1.112

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* All specimens broken in removing forms; strength interpolated from other tests.

TABLE 8.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 93).—*Continued.*

Reference No.	Mix by Volume.	Water.	Aggregate.			Time of Mixing, min.	Concrete.					
		Ratio to Cement by Volume.	Range in Size.	Weight, lb. per cu. ft.	Density.		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
							7 days	28 days	3 mo.			
MACHINE-MIXED CONCRETE.—*Continued.*												
55	1:5	1.51	0–8	110.0	0.660	10	280	580	1080	134.0	0.688	1.101
56						5	230	580	1200	131.8	0.677	1.118
57						2	180	480	920	131.1	0.672	1.126
58						1	140	470	1060	132.0	0.677	1.118
59						½	80	250	490	129.5	0.665	1.139
60						¼	30	*100	230	127.5	0.654	1.157
Aver....							160	410	830	131.0	0.672	1.126
61	1:5	1.43	0–4	115.0	0.690	10	370	760	1400	137.0	0.710	1.108
62						5	310	670	1620	133.0	0.690	1.140
63						2	180	520	1060	134.9	0.700	1.124
64						1	200	650	1630	135.5	0.703	1.119
65						½	110	220	610	133.2	0.691	1.138
66						¼	50	180	270	131.8	0.684	1.150
Aver....							200	500	1100	134.2	0.696	1.130
67	1:5	1.25	0–⅜	122.5	0.741	10	500	1020	2030	141.8	0.753	1.114
68						5	460	1090	2100	144.5	0.767	1.093
69						2	310	1000	1970	143.6	0.762	1.100
70						1	280	1000	2200	142.5	0.757	1.108
71						½	230	760	1590	141.0	0.748	1.120
72						¼	160	500	1090	141.7	0.752	1.115
Aver....							320	890	1830	142.5	0.756	1.108
73	1:5	1.08	0–¾	126.5	0.759	10	840	1700	2830	150.8	0.812	1.053
74						5	510	1590	3010	149.6	0.806	1.062
75						2	460	1390	2900	150.5	0.811	1.055
76						1	380	1300	2640	147.9	0.796	1.074
77						½	360	1000	2400	147.5	0.794	1.077
78						¼	250	840	1840	146.8	0.791	1.082
Aver....							470	1300	2600	148.8	0.802	1.067
79	1:5	0.82	0–2	128.5	0.771	10	1410	2300	3290	157.2	0.867	1.002
80						5	980	2260	3540	156.6	0.863	1.006
81						2	790	2020	3320	156.7	0.864	1.005
82						1	740	2000	3260	155.5	0.858	1.012
83						½	560	1780	2700	155.0	0.855	1.016
84						¼	600	1560	2810	149.8	0.827	1.051
Aver....							850	1990	3150	155.1	0.856	1.015

* All specimens broken in removing forms; strength interpolated from other tests.

TABLE 8.—A STUDY OF THE TIME OF MIXING CONCRETE (SERIES 93).—*Continued.*

Reference No.	Time of Mixing min.	Concrete.					
		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
		7 days	28 days	3 mo.			
GRAND AVERAGE OF ALL MIXES AND AGGREGATES—MACHINE-MIXED CONCRETE.							
1, 7, 13, 19, 79	10	1010	1860	2800	148.6	0.799	1.068
2, 8, 14, 20, 80	5	830	1750	2880	147.5	0.794	1.077
3, 9, 15, 21, 81	2	650	1540	2630	147.5	0.793	1.077
4, 10, 16, 22, 82	1	480	1440	2450	146.7	0.790	1.082
5, 11, 17, 23, 83	½	430	1200	2200	145.6	0.784	1.090
6, 12, 18, 24, 84	¼	380	1080	1950	144.8	0.779	1.097
GRAND AVERAGE OF 0–1¼ AGGREGATES AND ALL MIXES—MACHINE-MIXED CONCRETE.							
1, 7, 13, 19, 25, 31, 37, 43	10	1320	2400	3560	153.8	0.837	1.062
2, 8, 14, 20, 26, 32, 38, 44	5	1120	2230	3490	152.6	0.832	1.069
3, 9, 15, 21, 27, 33, 39, 45	2	880	1970	3220	152.3	0.830	1.070
4, 10, 16, 22, 28, 34, 40, 46	1	600	1800	2870	151.6	0.827	1.075
5, 11, 17, 23, 29, 35, 41, 47	½	580	1580	2830	150.9	0.822	1.079
6, 12, 18, 24, 30, 36, 42, 48	¼	520	1480	2620	150.5	0.821	1.083
GRAND AVERAGE OF 1–5 MIXES AND ALL AGGREGATES—MACHINE-MIXED CONCRETE.							
19, 49, 55, 61, 67, 73, 79	10	720	1350	2150	143.8	0.761	1.074
20, 50, 56, 62, 68, 74, 80	5	550	1270	2280	142.6	0.755	1.084
21, 51, 57, 63, 69, 75, 81	2	380	1030	1940	142.8	0.755	1.082
22, 52, 58, 64, 70, 76, 82	1	330	1050	2000	142.1	0.752	1.087
23, 53, 59, 65, 71, 77, 83	½	270	810	1580	140.7	0.745	1.097
24, 54, 60, 66, 72, 78, 84	¼	220	680	1360	139.4	0.738	1.109

HAND-MIXED CONCRETE.

Hand-mixed concrete cylinders were made at the same time as the machine-mixed batch of the same composition. The same materials were used. Specimens made by the same men, and stored and tested under identical conditions.

Each cylinder was proportioned and mixed in a separate batch.

These specimens bear Reference No. 200 higher than the machine-mixed concrete of the same kind. The making of the 4 cylinders in a set was distributed over the same periods as the machine-mixed concrete.

Reference No.	Mix by Volume.	Water.	Aggregate.			Time of Mixing, min.	Concrete.					
		Ratio to Cement by Volume.	Range in Size.	Weight, lb. per cu. ft.	Density.		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
							7 days	28 days	3 mo.			
201	1:2	0.50	0–1¼	130.0	0.780	..	1560	3380	5060	151.9	0.792	1.290
207	1:3	0.70	0–1¼	130.0	0.780	..	1300	2780	4200	152.4	0.814	1.155
213	1:4	0.81	0–1¼	130.0	0.780	..	940	2180	3660	153.0	0.828	1.087
219	1:5	0.91	0–1¼	130.0	0.780	..	760	1950	3480	154.1	0.843	1.040
225	1:6	1.01	0–1¼	130.0	0.780	..	580	1670	3090	154.4	0.851	1.012
231	1:7	1.10	0–1¼	130.0	0.780	..	530	1430	2550	153.8	0.852	0.997
237	1:9	1.32	0–1¼	130.0	0.780	..	360	1030	1870	152.5	0.850	0.983
243	1:15	1.94	0–1¼	130.0	0.780	..	190	440	800	146.8	0.826	0.983
Aver....	....	....	....	....	....	......	780	1860	3090	152.4	0.832	1.068
249	1:5	1.57	0–14	104.0	0.624	..	140	380	870	128.8	0.653	1.104
255	1:5	1.51	0–8	110.0	0.660	..	160	500	950	132.2	0.679	1.116
261	1:5	1.43	0–4	115.0	0.690	..	180	600	1320	136.2	0.709	1.114
267	1:5	1.25	0–⅜	123.5	0.741	..	270	900	1720	143.0	0.760	1.104
273	1:5	1.08	0–¾	126.5	0.759	..	440	1260	2550	151.0	0.815	1.056
219	1:5	0.91	0–1¼	130.0	0.780	..	760	1950	3480	154.1	0.843	1.040
279	1:5	0.82	0–2	128.5	0.771	..	670	2000	3500	156.5	0.863	1.006
Aver ...	....	....	....	....	....	......	370	1080	2060	143.1	0.760	1.077

that is, the volume of water, in terms of the volume of the cement in the batch using 1 sack of cement (94 lb.) as the equivalent of 1 cu. ft.

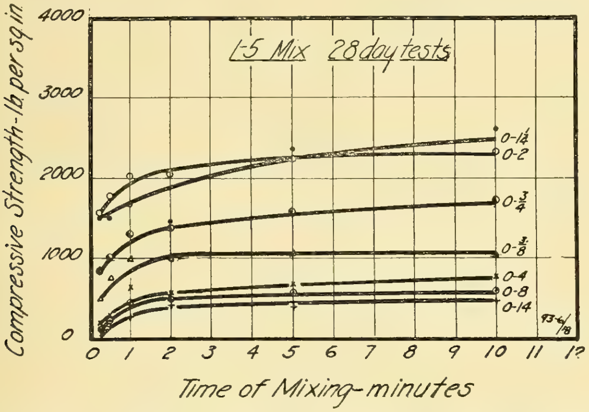

FIG. 25.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE. Series 93—1:5 mix, 28-day tests, different gradings of aggregate. For similar tests at 3 months see Fig. 27. Compare Fig. 4, 21, 27, 29.

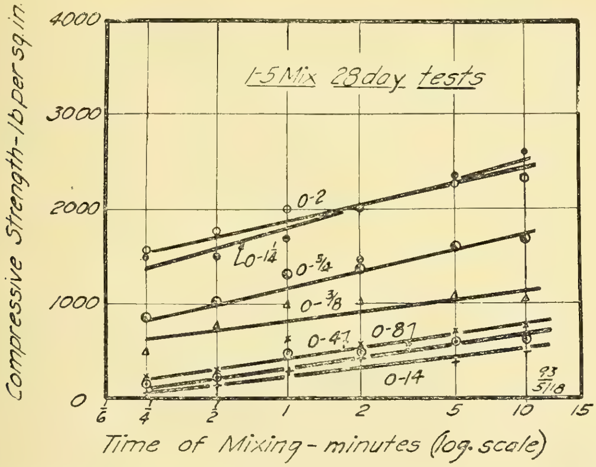

FIG. 26.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE. Series 93—Same data as in Fig. 25, except time of mixing platted to logarithmic scale. Compare Fig. 5, 6, 22, 28, 30.

In Series 89 the mix and grading of aggregate were maintained constant, but the water content was varied from a rather stiff mix to a very wet one. In this series both compression and wear tests were made. The influence of water on the strength of concrete in this series is shown in Fig. 7. It is seen that the strength decreases with the water content. If the values are platted to a logarithmic scale of strengths as in Fig. 8, this relation is expressed by a straight line. The path of these curves is expressed by an equation of the form,

$$S = \frac{A}{B^x} \dots\dots\dots\dots\dots\dots (1)$$

Where S = compressive strength at a given age, x = water ratio of the concrete.

A and B are constants for a given cement and given conditions of test.

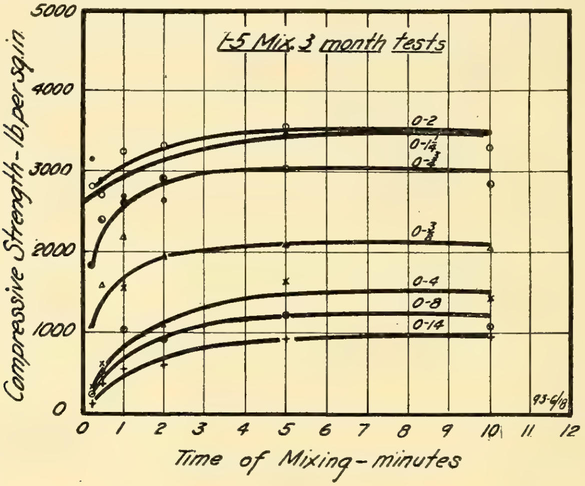

FIG. 27.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.
Series 93—3-mo. tests, similar to Fig. 25.
Compare Fig. 4, 21, 25, 29.

A similar set of curves is platted in Fig. 10, a separate curve being drawn for each age. The change in water content in this series was made in accordance with method (1) mentioned above.

In Series 93 the water content was varied on account of differences in the mix, using aggregate of a given grading, and also due to differences in the grading, using concrete of a given mix. In both instances the concrete was mixed to the same plasticity. The influence of water on the compressive strength in this series is shown in Fig. 31–34. Separate curves are drawn for the hand-mixed and machine-mixed concrete. It will be noted here that exactly the same relation exists as was pointed out above in discussing Series 89. The variation in the water content in these tests was made in accordance with methods (2) and (3) mentioned above. Fig. 44, 45, 52 and 53 show the same influence of water on the strength of concrete from still other series of tests. The curves in Fig. 52 are sensibly straight lines, since the range of consistencies is very narrow. It is the narrow range in consistencies which has been generally used in tests that has led to the erroneous conclusion that the strength of concrete is a linear function of the amount of water used. The relation is, of course, sensibly a linear one for any narrow range in consistency. This means simply that the tangent to any point of the curves in Fig. 7, 9, etc., is sensibly the same as the curve, if only a short arc is considered. The true relation for all parts of the range is given by equation (1).

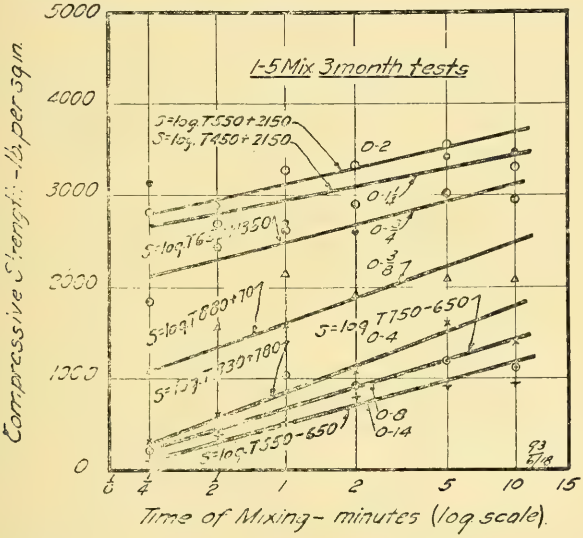

FIG. 28.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 27, except time of mixing platted to logarithmic scale.
The equations require that time of mixing be expressed in seconds.
Compare Fig. 5, 6, 22, 26, 30.

The water content expressed as a fraction of the volume of the cement in the batch is the only satisfactory criterion for strength of concrete made up of given materials mixed in widely different proportions. It will be seen from the tests included in this report that it *makes no difference why the water content is changed; the result is exactly the same in the end.* This change in water may be due to changing the plasticity of the mix or the use of richer mixes or aggregates of different gradings, or combinations of these factors. The influence of water on the strength of concrete, discussed above, has

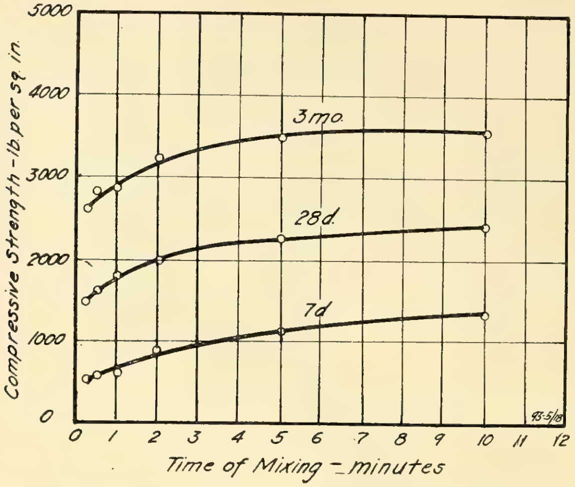

FIG. 29.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.
Series 93—Grand average for all times of mixing for 0–1¼ in. aggregates, regardless of mix.
Compare Fig. 4, 21, 23, 27.

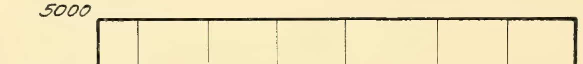

FIG. 30.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.
Series 93—Same data as in Fig. 29; time of mixing platted to logarithmic scale.
Equations require that time of mixing be expressed in seconds.
Compare Fig. 5, 6, 22, 26, 28.

TABLE 9.—EFFECT OF RATE OF ROTATION OF MIXER DRUM (SERIES 96).

Compression tests of 6 by 12-in. concrete cylinders.
Smith tilting "Mascot" mixer; capacity 3½ cu. ft.
Time of mixing 1 minute.
Portland cement—a mixture of equal parts of 4 brands purchased in Chicago market.
Aggregates—sand and pebbles from Elgin, Ill.; graded alike for all batches; weight 130.0 lb. per cu. ft.; density 0.780.
Mixing was begun after all materials were in the drum.
Specimens made in steel forms.
Cylinders stored in damp sand; tested damp.
Each value is the average from tests of 4 cylinders from the same batch.
All values in a horizontal line are from a single batch of concrete.
16 cylinders were made from each batch; the 1st, 5th, 9th and 13th were tested at 7 days; 2d, 6th, 10th and 14th at 28 days; etc.

Group.	Reference No.	Date Mixed (1918).	Mix by Volume.	Water.		Size of Aggregate.	R.P.M. of Mixer Drum.	Concrete.					
				Relative Consistency.	Ratio to Cement by Volume.			Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
								7 days	28 days	3 mo.			
1	7	1/23	1:5	100	0.83	0–1¼	8	1130	2570	3870	154.7	0.850	1.030
	8						15	1200	2660	4460	155.3	0.855	1.025
	9						18	1250	2640	4240	155.9	0.856	1.022
	10						21	1120	2630	4340	155.6	0.855	1.023
	11						25	1050	2800	4150	155.5	0.855	1.023
	12						30	1000	2720	3880	155.2	0.855	1.025
Aver.								1120	2670	4160	155.4	0.854	1.025
2	1	1/22	1:5	110	0.91	0–1¼	8	680	1830	2910	153.1	0.840	1.046
	2						15	740	2140	3410	154.3	0.844	1.039
	3						18	790	2340	3800	154.4	0.845	1.038
	4						21	740	2210	3560	153.8	0.841	1.042
	5						25	680	2000	2800	154.4	0.845	1.038
	6						30	680	1940	2830	153.9	0.843	1.041
Aver.								720	2080	3220	154.0	0.843	1.041
3	13	1/24	1:5	110	0.91	0–1¼	8	620	1620	2550	153.7	0.842	1.041
	14						15	700	1880	3370	155.2	0.849	1.032
	15						18	820	2050	3420	155.3	0.850	1.031
	16						21	760	2200	3490	155.1	0.849	1.032
	17						25	820	2150	3560	155.0	0.848	1.033
	18						30	740	1670	3380	155.2	0.850	1.031
Aver.								740	1930	3300	154.9	0.848	1.033
4	31	1/26	1:5	110	0.91	0–1¼	8	660	1790	2770	153.5	0.840	1.043
	32						15	840	2160	3230	154.4	0.845	1.038
	33						18	850	2110	3290	155.1	0.850	1.032
	34						21	820	1880	2980	155.0	0.849	1.033
	35						25	850	1980	3180	154.7	0.846	1.037
	36						30	950	2160	3040	155.6	0.851	1.030
Aver.								830	2010	3080	154.7	0.847	1.036

TABLE 9.—EFFECT OF RATE OF ROTATION OF MIXER DRUM (SERIES 96).—*Continued.*

Group.	Reference No.	Date Mixed (1918).	Mix by Volume.	Water.		Size of Aggregate.	R.P.M. of Mixer Drum.	Concrete.					
				Relative Consistency.	Ratio to Cement by Volume.			Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
								7 days	28 days	3 mo.			
5	19	1/25	1:5	125	1.04	0–1¼	8	480	1290	2420	152.7	0.827	1.060
	20						15	540	1470	2670	153.1	0.831	1.056
	21						18	610	1500	2940	154.5	0.838	1.047
	22						21	540	1390	3120	153.4	0.832	1.054
	23						25	610	1510	2810	153.8	0.834	1.051
	24						30	540	1300	2660	154.0	0.835	1.050
Aver.								560	1410	2770	153.6	0.833	1.053
6	25	1/28	1:5	150	1.21	0–1¼	8	310	880	1580	151.7	0.809	1.082
	26						15	300	850	1650	153.1	0.816	1.072
	27						18	350	920	1800	153.0	0.815	1.073
	28						21	360	1020	1760	152.2	0.814	1.078
	29						25	380	1020	1970	153.2	0.818	1.071
	30						30	480	1140	1920	153.0	0.816	1.074
Aver.								360	970	1780	152.7	0.815	1.075

Reference No.	R.P.M. of Mixer Drum.	Concrete.					
		Compressive Strength, lb. per sq. in.			Weight, lb. per cu. ft.	Density.	Yield.
		7 days	28 days	3 mo.			
GRAND AVERAGE OF ALL CONSISTENCIES.							
1– 7–13–19–25–31	8	650	1660	2680	153.2	0.835	1.050
2– 8–14–20–26–32	15	720	1860	3130	154.2	0.840	1.044
3– 9–15–21–27–33	18	780	1930	3250	154.7	0.842	1.040
4–10–16–22–28–34	21	720	1890	3210	154.2	0.840	1.044
5–11–17–23–29–35	25	730	1910	3080	154.4	0.841	1.042
6–12–18–24–30–36	30	730	1820	2950	154.5	0.842	1.042
GRAND AVERAGE OF 110 PER CENT CONSISTENCY.							
1–13–31	8	650	1750	2740	153.4	0.841	1.043
2–14–32	15	760	2060	3340	154.6	0.846	1.036
3–15–33	18	820	2170	3500	154.9	0.848	1.030
4–16–34	21	770	2100	3340	154.6	0.846	1.036
5–17–35	25	780	2040	3180	154.7	0.846	1.036
6–18–36	30	790	1920	3080	154.9	0.848	1.034
GRAND AVERAGE OF ALL 110 PER CENT CONSISTENCY AND ALL R.P.M.							
1 to 6, 13 to 18, and 31 to 36		760	2010	3200	154.5	0.846	1.037

been brought out in the same way by probably a dozen different series of tests carried out in this Laboratory. In one series the water content as compared with the volume of the cement varied from 0.35 to 4.00—a much wider range than was covered by the tests herein reported. See Engineering News-Record, May 2, 1918.

The constants in equation (1) may be determined directly from the diagrams given in Fig. 8, 10, 32, 34, 45 and 53. The constant A is determined by projecting the curve to the left to intersection with the ordinate at zero water content. Log B is the slope of the straight line derived by platting the strengths to a logarithmic scale.

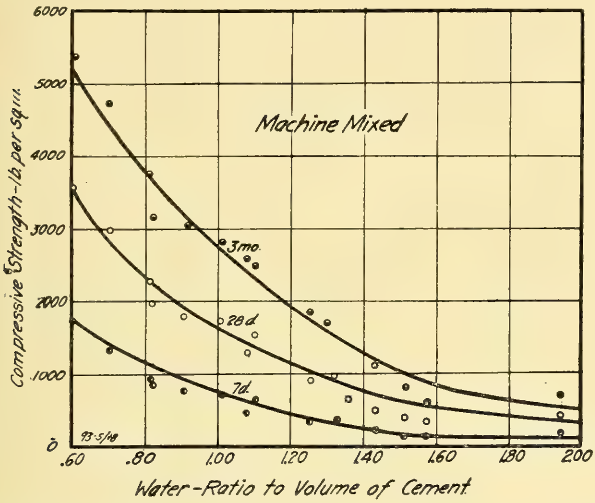

FIG. 31.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE

Series 93—Includes all tests of machine-mixed concrete in Series 93. Each point is the average of 6 different times of mixing for each mix and grading.

Compare Fig. 7, 9, 33, 44, 52.

Equation (1) given above may be considered as expressing the law of concrete strength for variations in the proportions of given lots of cement, aggregate and water. This law is subject to the following limitations:

(*a*) Mix not so wet that the mixture of cement and aggregates cannot retain all the water.

(*b*) Mix not too dry for maximum strength for the condition under which the concrete is placed. (That is, the concrete is in a plastic condition.)

(*c*) Aggregate grading not too coarse for maximum strength with the quantity of cement and water used.

Of these limitations (*a*) is the only one which is usually encountered in concrete work. In mixes so wet that all the water is not retained the strength will be a little higher than the equation indicates. However, this condition does not prevail until mixes are reached which are so wet that extremely low strength will be secured anyhow. Before a mix becomes too dry for maximum strength it is so harsh-working that this limitation is not encountered except in concrete products plants where machine molding is done. It is seldom that the aggregates are graded too coarse for the limitations of this law, for the same reason as given for the dry mixes.

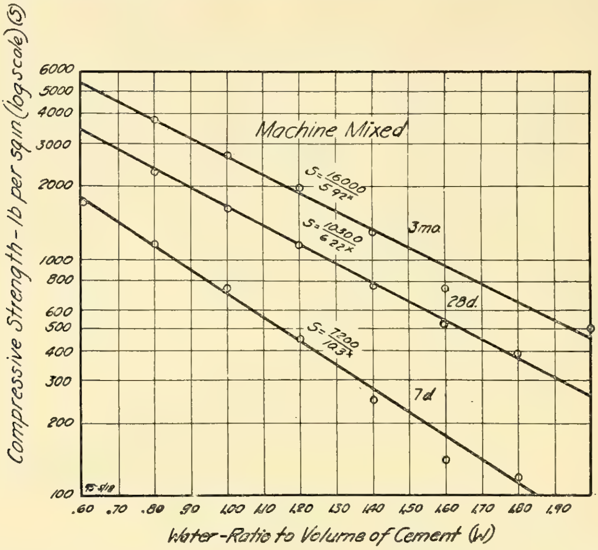

FIG. 32.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 31, except strength platted to logarithmic scale. The points are platted rom the curves in Fig. 31.

Compare Fig. 8, 10, 34, 45, 53.

The tests covered in this report give about the following values as showing the influence of water on the compressive strength of concrete:

$$\text{7 days} \quad S = \frac{9200}{14.0^x} \ldots\ldots\ldots\ldots (3)$$

$$\text{28 days} \quad S = \frac{14000}{8.2^x} \ldots\ldots\ldots\ldots (4)$$

$$\text{3 months} \quad S = \frac{16600}{5.9^x} \ldots\ldots\ldots\ldots (5)$$

TABLE 10.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE (SERIES 97).

Compression tests of 6 by 12-in. cylinders.
1:4 mix by volume.
Portland cement—equal parts of 4 brands purchased in Chicago market.
Aggregates—sand and pebbles from Elgin, Ill., graded alike (0–1¼ in.) for all batches
Smith "Mascot" mixer; 3½ cu. ft. capacity.
Each batch mixed 1 min.—18 revolutions.
Mixing was begun after all materials were in the drum.
Specimens made in steel forms.
Cylinders stored in damp sand; tested damp.
Each value is the average from tests of 5 cylinders from the same batch.
All values in a horizontal line are from a single batch of concrete.
15 cylinders were made from each batch; the 1st, 4th, 7th, 10th and 13th were tested at 7 days; the 2d, 5th, 8th, 11th and 14th, at 28 days, etc.

Reference No.	Date Mixed (1918).	Temperature, deg. C.				Test for Consistency.	Compressive Strength.					
							lb. per sq. in.			Mean Error, per cent.		
		Mixing Water.	Aggregate.	Concrete.	Air.	Penetration, in.	7 days	28 days	3 mo.	7 days	28 days	3 mo.
RELATIVE CONSISTENCY, 90 PER CENT. WATER, 0.68 OF VOLUME OF CEMENT.												
1	1/31	0	13.0	12.3	13.5	1.0–1.3	1420	3660	4940	7.4	3.6	7.3
2	2/7	0	14.0	13.5	15.0	1.0–2.0	1680	3250	4750	4.1	5.4	6.1
3	1/31	10	13.0	13.5	14.0	1.0–1.3	1450	3250	4990	4.1	8.7	3.2
4	2/7	10	14.0	14.5	15.0	1.0–2.0	1870	3540	4820	5.0	5.6	3.4
5	1/31	20	13.0	15.5	15.0	1.0–1.3	1470	3440	5050	10.8	11.6	2.4
6	2/7	20	14.0	16.0	15.0	1.0–2.0	1620	3290	4590	4.7	3.8	13.2
7	1/31	30	13.0	17.0	15.5	1.0–1.3	1490	3660	5130	13.9	11.1	2.0
8	2/7	30	14.0	18.2	15.0	1.0–2.0	2090	3610	4850	4.0	5.2	5.8
9	1/31	40	13.0	19.0	14.7	1.0–1.3	1490	3410	4830	5.8	8.6	6.9
10	2/7	40	14.0	19.8	15.2	1.0	2090	3560	4790	10.0	7.8	5.4
11	1/31	50	13.0	21.0	15.0	1.0–1.3	1450	3410	4630	6.9	6.7	7.0
12	2/7	50	14.0	22.5	15.5	1.0	1820	3390	4690	5.3	2.5	3.1
13	2/1	60	12.5	20.0	12.8	0.5–1.3	1680	3460	5130	6.1	10.7	6.8
14	2/7	60	14.0	23.0	15.7	1.0	1880	3630	4740	15.2	9.4	12.3
15	2/1	70	12.5	23.0	14.0	0.5–1.3	1600	3500	5030	8.1	9.6	4.7
16	2/7	70	14.0	25.5	15.8	1.0	2040	3540	4880	7.3	4.1	4.7
17	2/1	80	12.5	25.0	15.0	0.5–1.3	1550	3350	4710	14.5	8.2	6.0
18	2/7	80	14.0	27.0	16.5	1.0	2110	3800	4820	13.5	3.0	5.1
19	2/7	90	14.0	28.0	15.7	1.0	1810	3320	4450	10.8	7.3	10.2
20	2/9	90	15.5	28.0	15.5	1.0	2120	3520	4660	4.1	9.8	11.0
21	2/11	100	14.0	29.2	15.0	1.0	1970	2930	4320	3.6	12.2	7.4
21A	2/12	100	14.5	30.5	15.0	1.0	1540	3050	3480	9.4	13.1	13.3
Aver......			13.6	21.0	15.0	......	1740	3440	4740	7.9	7.6	6.7

TABLE 10.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE (SERIES 97).—*Continued.*

Reference No.	Date Mixed (1918).	Temperature, deg. C.				Test for Consistency.	Compressive Strength.					
							lb. per sq. in.			Mean Error, per cent.		
		Mixing Water.	Aggregate.	Concrete.	Air.	Penetration, in.	7 days	28 days	3 mo.	7 days	28 days	3 mo.
RELATIVE CONSISTENCY, 100 PER CENT. WATER, 0.76 OF VOLUME OF CEMENT.												
22	1/30	0	14.0	11.5	13.5	2.0–3.5	1120	3030	4580	3.7	3.4	5.7
23	2/6	0	12.5	11.0	14.0	2.0–2.5	1430	3310	4650	6.8	5.1	3.9
24	1/30	10	14.0	13.0	14.0	2.0–3.5	1200	3000	4370	3.5	3.1	12.1
25	2/6	10	12.5	13.0	14.0	2.0–4.0	1240	3070	4420	8.3	3.5	3.7
26	1/30	20	14.0	15.5	14.5	2.0–3.5	1170	3010	4270	6.6	3.4	5.3
27	2/6	20	12.5	15.5	14.2	2.0–5.0	1330	3150	4490	2.5	3.5	4.3
28	1/30	30	14.0	17.6	14.5	2.5–5.0	1260	3000	4390	5.9	6.4	8.2
29	2/6	30	12.5	18.0	14.2	2.0–3.0	1260	2980	4390	7.3	2.6	4.4
30	1/30	40	14.0	19.5	14.5	2.5–5.5	1210	2900	4500	3.5	5.2	4.9
31	2/6	40	12.5	20.5	14.2	2.0–4.0	1540	3240	4610	7.2	4.7	6.1
32	1/30	50	14.0	22.0	14.8	2.0–3.0	1150	2990	4430	3.5	2.4	2.8
33	2/6	50	12.5	22.5	14.7	2.0–2.5	1690	3400	4580	8.8	3.8	10.2
34	1/30	60	14.0	24.2	15.0	2.0–3.5	1180	3100	4220	7.4	4.3	6.9
35	2/6	60	12.5	23.8	14.7	1.0–2.0	1530	3340	4680	11.5	5.7	4.1
36	1/30	70	14.0	27.0	15.3	2.0–4.0	1210	3100	4120	7.1	10.7	11.2
37	2/6	70	12.5	27.0	15.0	1.0–2.5	1480	3160	4510	8.6	7.6	4.9
38	1/30	80	14.0	28.5	15.5	1.0–1.5	1200	3050	4230	15.8	1.9	8.5
38A	2/6	80	12.5	28.3	15.3	1.0–2.0	1480	3090	4180	7.2	2.8	8.9
40	2/6	90	12.5	31.0	15.7	1.0–2.0	1170	2870	3980	19.6	11.6	4.5
41	2/9	90	15.5	29.0	15.5	1.0	1990	3460	4180	5.4	4.0	11.7
42	2/11	100	14.0	30.0	15.0	1.0	1830	3160	4520	3.2	6.2	6.9
42A	2/12	100	14.5	33.5	15.2	1.0–2.0	1610	2690	4410	7.5	9.0	8.1
Aver.			13.4	21.9	14.7	....	1360	3060	4360	7.3	5.0	6.7

TABLE 10.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE (SERIES 97).—*Continued.*

Reference No.	Date Mixed (1918).	Temperature, deg. C.				Test for Consistency.	Compressive Strength.					
		Mixing Water.	Aggregate.	Concrete.	Air.	Penetration, in.	lb. per sq. in.			Mean Error, per cent.		
							7 days	28 days	3 mo.	7 days	28 days	3 mo.
RELATIVE CONSISTENCY, 110 PER CENT. WATER, 0.83 OF VOLUME OF CEMENT.												
43	1/29	0	12.0	9.5	13.0	4.0–8.0	960	2720	4340	5.3	2.3	6.2
44	2/4	5	9.0	9.5	9.5	4.0–7.0	1050	2720	4270	4.8	6.9	1.8
45	1/29	10	12.0	12.0	13.0	4.0–8.0	1110	2940	4590	5.6	4.7	1.2
46	2/4	15	9.0	11.5	9.5	4.0–7.0	1000	2610	3860	9.5	9.4	8.5
47	1/29	20	12.0	15.0	13.0	4.0–8.5	980	2580	4010	5.6	3.7	6.9
48	2/4	25	9.0	14.0	9.5	4.0–8.0	1000	2790	3830	7.7	7.8	7.6
49	1/29	30	12.0	17.2	13.0	4.0–8.5	1000	2560	4120	2.3	4.7	5.8
50	2/4	35	9.0	17.0	9.0	4.0–7.5	1190	2730	3870	11.4	2.0	2.4
51	1/29	40	12.0	20.0	13.0	4.0–8.5	1130	2740	4440	6.2	9.3	3.6
52	2/4	45	9.0	19.8	8.8	4.0–9.0	1180	2420	4210	8.5	9.4	7.8
53	1/29	50	12.0	22.3	13.0	4.0–7.0	1000	2920	4290	11.0	3.8	6.1
54	2/4	50	9.0	22.0	8.5	4.5–8.0	1050	2610	3810	3.8	13.0	7.0
55	1/29	60	12.0	24.5	13.0	4.0–7.0	1040	2720	4300	6.0	4.8	7.6
56	2/4	60	9.0	23.5	10.0	3.0–6.0	1050	2550	3710	7.9	7.8	10.1
57	1/29	70	12.0	27.0	13.5	3.0–6.0	1090	3010	4370	12.3	10.6	10.2
58	2/4	70	9.0	26.0	10.5	3.0–4.5	1130	2700	4030	15.1	6.2	5.2
59	1/29	80	12.0	31.0	13.7	3.0–6.0	960	2450	3850	7.0	7.4	9.6
59A	2/4	80	9.0	28.2	10.8	2.0–3.0	1170	2610	3810	13.6	4.1	16.6
60	1/29	85	12.0	34.0	13.7	3.0–6.0	1170	2700	4190	10.3	14.5	7.8
61	2/4	91	9.0	32.0	11.2	2.0–3.5	1110	2890	3840	2.9	9.7	5.9
62	2/9	90	15.5	30.8	15.5	4.0–6.0	1320	2520	3820	7.4	5.3	14.3
63	2/11	100	14.0	34.8	15.0	1.0–2.0	1520	2880	4240	3.9	4.6	9.9
63A	2/12	100	14.5	34.7	15.0	2.0–3.5	1480	3080	4300	6.2	4.5	9.0
Aver.			11.0	22.5	11.9	....	1120	2720	4090	7.6	6.8	7.4

TABLE 10.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE (SERIES 97).—*Continued.*

Reference No.	Date Mixed (1918).	Temperature, deg. C.				Test for Consistency.	Compressive Strength.					
							lb. per sq. in.			Mean Error, per cent.		
		Mixing Water.	Aggregate.	Concrete.	Air.	Penetration, in.	7 days	28 days	3 mo.	7 days	28 days	3 mo.
RELATIVE CONSISTENCY, 125 PER CENT. WATER, 0.95 OF VOLUME OF CEMENT.												
64	2/5	0	10.5	9.0	11.5	3.0– 4.5	900	2550	3980	6.3	3.4	12.6
64A	2/8	0	15.0	12.0	16.0	8.0–10.0	890	2100	3430	8.8	5.0	5.5
65	2/5	10	10.5	10.5	11.5	8.0–11.0	740	2080	3300	10.4	15.9	7.6
65A	2/8	10	15.0	13.0	16.0	8.0–10.0	890	1860	3280	5.5	6.7	7.1
66	2/5	20	10.5	14.0	12.0	8.0–11.0	720	1830	3160	4.3	12.6	9.2
66A	2/8	20	15.0	16.5	16.0	8.0–10.0	850	1710	3250	11.8	8.5	8.8
67	2/5	30	10.5	15.5	11.7	8.0–11.0	810	2080	3560	10.4	4.3	8.8
67A	2/8	30	15.0	19.0	16.0	8.0–11.0	1030	1900	3320	7.9	12.8	7.8
68	2/5	40	10.5	19.0	12.0	8.0–11.0	840	1990	3310	9.1	9.8	5.5
68A	2/8	40	15.0	22.0	16.0	8.0–11.0	1120	2170	3760	5.8	4.5	6.5
69	2/5	50	10.5	22.2	12.0	8.0–11.0	990	2410	3680	8.2	12.4	2.5
69A	2/8	50	15.0	24.5	16.0	8.0–11.0	1170	2210	3610	8.4	8.4	10.3
70	2/5	60	10.5	25.3	12.3	8.0–11.0	1070	2500	3660	3.7	11.8	8.3
70A	2/8	60	15.0	27.5	16.0	8.0–11.0	1030	2280	3450	5.7	7.1	3.8
71	2/5	70	10.5	28.0	12.3	8.0–11.0	880	2200	3430	8.7	9.5	1.6
71A	2/8	70	15.0	30.5	16.0	8.0–11.0	1160	2310	3620	12.4	7.6	5.7
72	2/5	80	10.5	31.0	12.5	8.0–11.0	1010	2540	3740	7.0	5.5	5.3
72A	2/8	80	15.0	32.0	16.5	8.0–11.0	820	1790	2790	7.2	6.3	7.2
73	2/5	90	10.5	33.5	12.5	8.0–11.0	980	2610	3910	11.8	4.6	6.9
73A	2/8	90	15.0	36.0	16.7	8.0–11.0	980	2180	2800	7.6	7.3	12.1
74	2/11	100	14.0	33.7	14.5	5.0– 9.0	1050	2210	3300	7.9	4.0	11.0
74A	2/12	100	14.5	35.5	15.0	7.0–10.0	920	2100	3480	5.1	10.6	5.3
Aver			12.9	23.2	14.3	....	950	2160	3450	7.9	8.1	7.2

The factor A does not change much with the age of the concrete. Certain series of tests not included in this report suggest that for a given cement A may be independent of the age of the concrete. The value of A does, however, depend on the quality of the cement. B varies widely with the age of the concrete, time of mixing and other conditions.

If in equation (4) $x = 1.00$ (that is, 1 cu. ft. water to 1 sack of cement, due allowance being made for water held or absorbed by aggregate) we find $S = 1710$ lb. per sq. in.; if $x = 0.77$ ($8.2^x = 5.06$), $S = 2770$ lb. per sq. in.

Let us compute by this means the effect of using 1 lb. (1 pint) more or less water than necessary in a 1-bag batch of concrete. Assume $x = 0.77$, which is about the quantity of water which should be used in concrete for road construction (that is, about 5.75 gallons per 1-bag batch). We found above that $x = 0.77$ gives a strength for the average conditions of these tests of 2770 lb. per sq. in. 1 lb. of water $= .016$ cu. ft. For an increase of 1 lb. we should then have $x = 0.786$, $8.2^x = 5.25$, and $S = 2670$ lb. per sq. in., a decrease in strength of 100 lb. per sq. in. or about 3 per cent. With this mix and the same consistency we may say that for small changes in cement content the strength is proportional to the quantity of cement in the batch. We are led then to the amazing conclusion that *1 pint more water than necessary* in a 1-bag batch produces the same effect as if we should

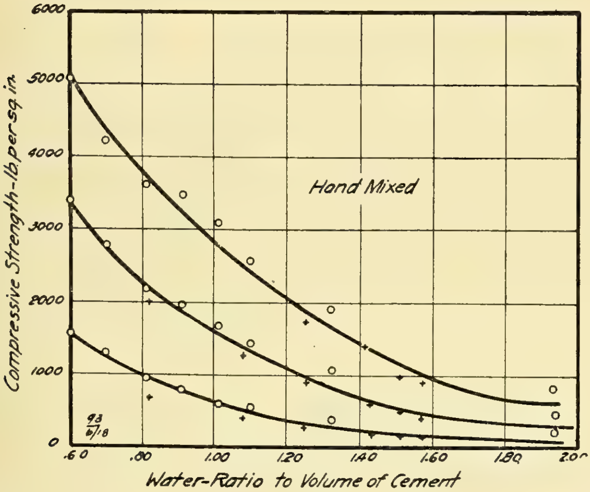

FIG. 33.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.
Series 93—Hand-mixed concrete in Table 8. Each value is the average of 4 tests made on different days. Compare Fig. 7, 9, 31, 44, 52.

leave out 3 lb. of cement. If a gallon more water than necessary is put in a 1-bag batch it means that approximately 25 per cent. of the cement is thereby wasted.

Table 11 shows the influence of water on the strength of concrete for a typical case. The values in the table are about what would be expected from a well-graded mix of 1–4 (or in usual terms 1–1½–3 or 1–2–3) concrete.

These tests give us an entirely new conception of the function performed by the various constituent materials. The use of a coarse, well-graded aggregate results in no gain in strength unless we take advantage of the fact that the amount of water necessary to produce a plastic mix can be reduced. In a similar way we may say that the use of more cement in a batch does not produce any beneficial effect except from the fact that a plastic, workable mix can be produced with relatively less water.

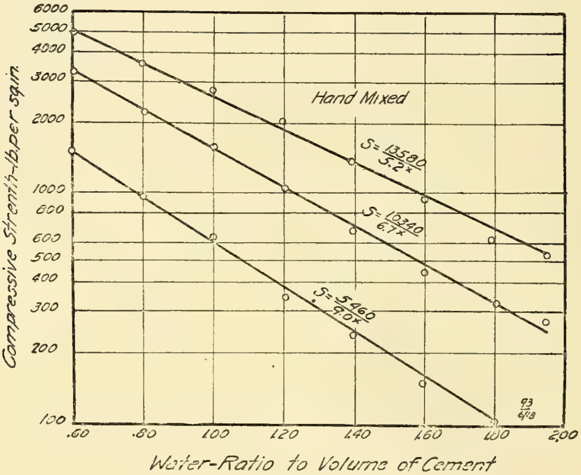

FIG. 34.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 33, except strength platted to logarithmic scale.
Compare Fig. 8, 10, 32, 45, 53.

The reason a rich mixture gives a higher strength than a lean one is not that more cement is used but because the concrete can be mixed (and usually is mixed) with a quantity of water which is relatively lower for the richer mixtures than for the lean ones. If advantage is not taken of the fact that *in a rich mix relatively less water can be used,* no benefit will be gained as compared with a leaner mix. In all this discussion the quantity of water is compared with the quantity of cement in the batch (cubic feet of water to 1 sack of cement) and not to the weight of the dry materials or of the concrete as is generally done.

The mere use of richer mixes has encouraged a feeling of security, whereas in many instances nothing more has been accomplished than wasting a large quantity of cement, due to the use of a great excess of mixing water. The universal acceptance of this false theory of concrete has exerted a most pernicious influence on the proper use of concrete materials and has proven to be an almost insurmountable barrier in the way of progress in the development of sound principles of concrete proportioning and construction.

TABLE 11.—EXAMPLE OF INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Values calculated from equation,

$$S=\frac{A}{B^x}=\frac{14000}{8.2^x}$$

Where S = Compressive strength
x = Water ratio
A and B are constants whose values depend on quality of cement and other conditions.
The water ratio is equivalent to the cubic feet of water to 1 sack (1 cu. ft.) of cement.

The strengths given in the table are about what would be expected from a 1:4 mix for usual consistencies, etc. The strength values are solely for comparative purposes in showing the influence of changing the water ratio. The table is based on the following assumptions: Mix 1:4, relative consistency 110 per cent, aggregate graded 0–1¼ in., mixed 1 min.

Water in a 1-bag Batch.		Relative Consistency, per cent.	Compressive Strength of Concrete at 28 Days.	
Gallons.	Water Ratio (x).		lb. per sq. in. (S).	Relative Strength, per cent.
5.75	.77	100	2770	100
6.0	.80	104	2600	94
6.25	.84	109	2400	87
6.5	.87	113	2250	81
7.0	.94	122	1950	70
7.5	1.01	131	1670	60
8.0	1.07	139	1470	53
9.0	1.21	157	1100	40
10.0	1.34	174	830	30
12.0	1.60	208	480	17
15.0	2.00	260	200	7

Rich mixes and well-graded aggregates are just as essential as ever, but we now have a proper appreciation of the true function of these materials, and a more thorough understanding of the injurious effect of too much water. Rich mixes and well-graded aggregates are, after all, only a means to an end, namely, to produce a plastic, workable concrete with a minimum quantity of water as compared with the cement used. Workability of concrete mixes is of fundamental significance. This factor is the only limitation which prevents the reduction of cement and water in the batch to much lower limits than are now practicable. The importance of any method of mixing, handling, placing and finishing concrete which will enable the builder to reduce the water content of the concrete to a minimum is at once apparent.

If equation (1) is interpreted literally we should be led to the conclusion that the concrete increases in strength as the water ratio is reduced up to the point where no water is used. This, of course, is not the case, since mixes so dry as to be non-plastic will show a falling off in strength. The quantity of water which will permit only a trace of moisture to be brought to the surface gives the highest strength. The value of water ratio which gives this result will depend upon the amount of cement (the mix), the grading of the aggregate, manipulation, etc. Definite maximum points are secured in the water-strength curves if the tests are carried to drier and drier mixes. From the maximum the strength falls off rapidly for either more or less water. The maximum strength occurs for usual conditions at a relative consistency of 85 to 90 per cent., based on the method of determining consistency used in these tests. For a typical water-strength curve, showing values for dry mixes, see Concrete Highway Magazine, April, 1917.

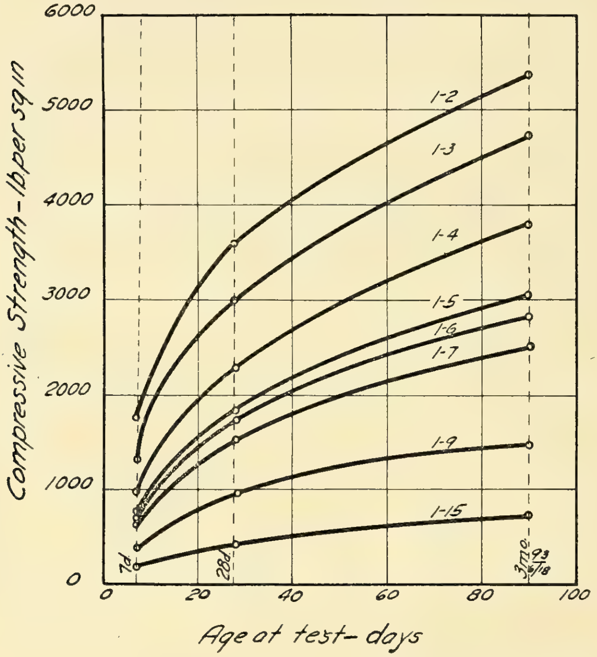

FIG. 35.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.

Series 93—Aggregates graded 0–1¼ in. Each point is the average of 24 tests from 6 times of mixing. Compare Fig. 19 and 38.

Few engineers or contractors fully realize the injurious effects produced by using too much mixing water in concrete. We frequently hear the following arguments in favor of the very wet mixes:

1. The excess water does no harm, since it runs off and evaporates.
2. While very wet concrete is weak at early ages, it gains in strength more rapidly than the drier mixes.
3. The richer mixes used in reinforced concrete buildings and in road construction are less affected by excess water than ordinary mixes.

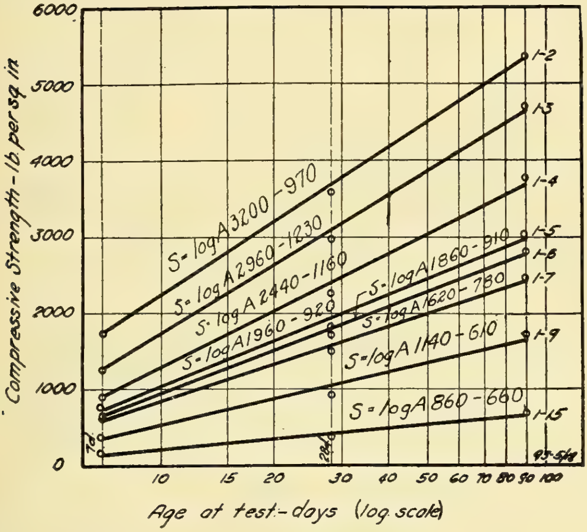

FIG. 36.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.
Series 93—Same data as in Fig. 35, except age is platted to logarithmic scale.
Compare Fig. 20, 37, 39, 46.

Let us examine these statements in view of the tests included in this report. The excess water does not run off until such a quantity has been used that all of it cannot be held by the concrete. This condition comes at about 200 per cent. relative consistency, that is, about double the water used for concrete of good quality. At this consistency we find a strength of about 20 per cent. of what should be secured. There was every opportunity for the water to run off in these tests. Some of the water evaporates, but it has already done the damage.

It was formerly believed that there was no truth in claim (2). It now appears from these tests that the wetter mixes do increase in strength somewhat more rapidly than the drier ones, but the difference in rate is not of sufficient importance to justify placing any confidence in this claim. A slight difference in rate cannot counteract a handicap of 300 to 400 per cent. in strength due to too much water in the original mix.

The exact interpretation of claim (3) hinges entirely on what constitutes excess water. For a given increase in quantity, that is, a certain number of pints, the richer mixes are more affected than the lean ones. This is shown by the steeper slopes of the curves in Fig. 9, etc., for the drier mixes or the mixes which for any reason have a lower water ratio. The effect which would be produced by a given change in plastic condition, for rich or lean mixes would depend on many other factors. There is no reason to believe that the conditions would ever be noticeably more favorable in the rich mixes.

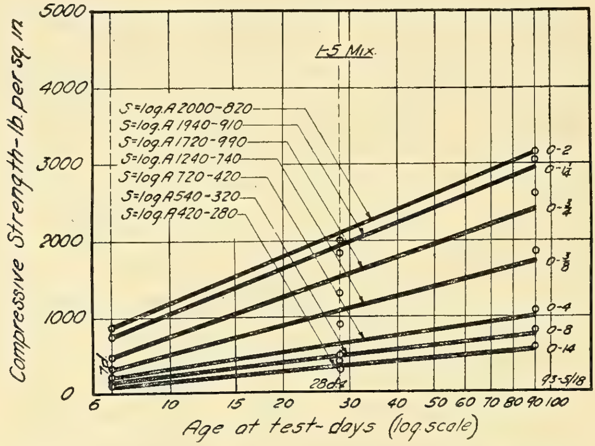

FIG. 37.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.

Series 93—1-5 mix, size of aggregate variable. Each point is the average of 24 tests based on 6 times of mixing for each grading. Age platted to logarithmic scale.

Compare Fig. 20, 36, 39, 46.

The only safe rule to follow with reference to water in concrete is to use the smallest quantity of mixing water which will give a plastic or workable mix, then provide plenty of moisture for the concrete during the period of curing which follows setting and hardening of the cement.

The reason for the injurious effect of too much water in concrete is undoubtedly found in the fact that the particles of cement are pushed farther and farther apart and from the aggregate particles to which they should adhere as water is added to the batch. Soon a condition is reached where practically no strength is obtained. If the water content is increased to 3 or 4 times the volume of the cement the strength is negligible (whether in neat cement or concrete). Beyond this range the mass never hardens. The cement is fully hydrated, but it offers no resistance to stress. In this instance the particles have been separated beyond the range of molecular action. These considerations furnish a method of determining the law of molecular attraction for a material of the nature of portland cement.

A brief discussion of the influence of water on the strength of concrete, based on a series of tests which is not included in this report

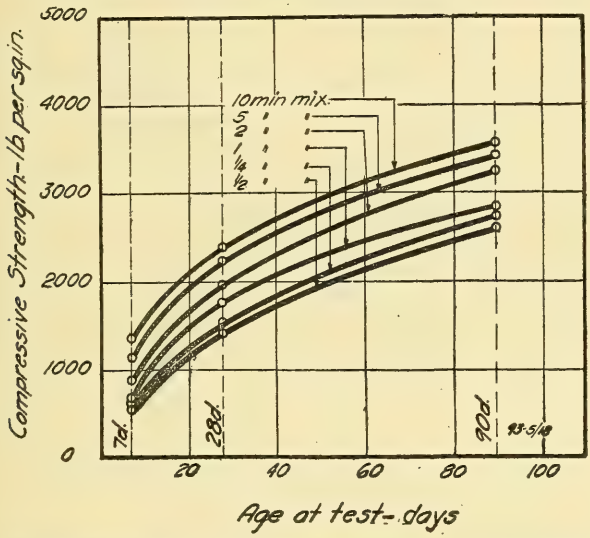

FIG. 38.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.
Series 93—All aggregates graded 0-1¼ in. Each point is the average of 32 tests—8 different mixes. Compare Fig. 19 and 35.

was given by the writer in Engineering News-Record for May 2, 1918, and in the Canadian Engineer for June 6, 1918. A more complete presentation of this work is being prepared for publication at an early date.

It is interesting to note that the strength of timber is affected by changes in water content in exactly the same manner as concrete. Any addition of moisture above the bone-dry condition causes a reduction in strength. The law of change in strength is of exactly the same form as our equation (1). This fact connotes a remarkable similarity in the molecular structure of timber and concrete. (For discussions of timber tests, see reports of U. S. Forest Service, and Forest

Products Testing Laboratory, or consult recent reference books on materials of construction.)

EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Tests were made on the effect of time of mixing for mixing periods of 15 sec. to 10 min. in both Series 89 and 93. In Series 89 the mix and the grading of the aggregate remained constant (1–4 mix, aggregate 0–1¼ in.) but the amount of mixing water varied from what we call 90 per cent. relative consistency to 200 per cent. This

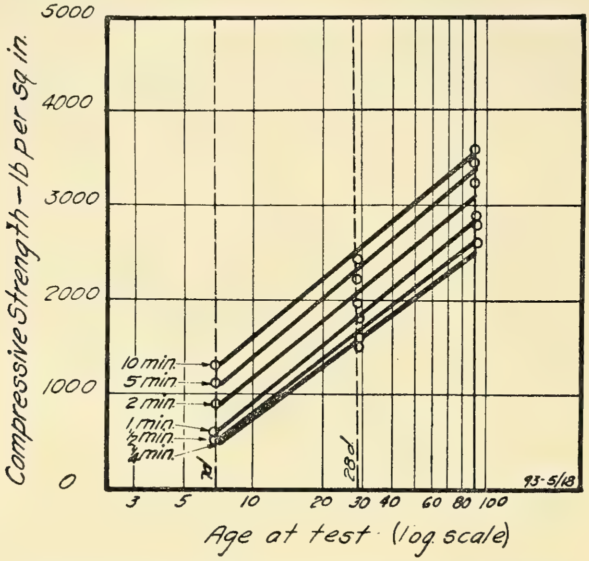

FIG. 39.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.

Series 93—Same data as in Fig. 38; age is platted to logarithmic scale. Compare Fig. 20, 36, 37, 46.

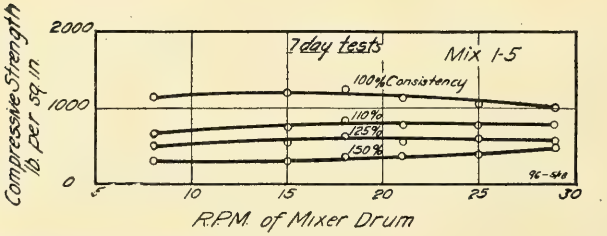

FIG. 40.—EFFECT OF RATE OF ROTATION OF MIXER DRUM ON STRENGTH OF CONCRETE.

Series 96—1:5 mix; 7-day tests. In general the values are the average of 4 tests. The 110 per cent consistency was mixed in triplicate on different days; each value on this curve is the average of 12 tests.

corresponds to a range in the water content from a rather stiff to a very wet mix, and a water ratio of 0.65 to 1.45. In Series 93 two variations in the proportions were made. In the first group of tests the grading of aggregates remained constant (0–1¼ in.) but the mix was varied from 1–2 to 1–15. In the second group in this series the mix remained constant (1–5) but the grading of the aggregate varied from sand 0–14, to a very coarse concrete aggregate, graded 0–2 in.

The effect of time of mixing concrete will be considered under the following sub-headings:

1. General discussion of time of mixing.
2. Concrete of different consistencies.
3. Concrete with different quantities of cement.
4. Concrete with aggregates of different sizes.
5. Further remarks on time of mixing.

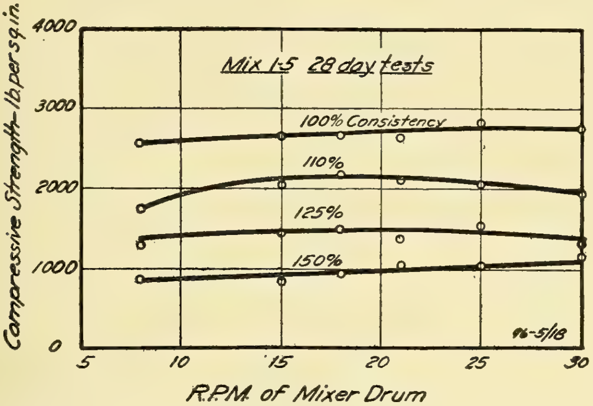

FIG. 41.—EFFECT OF RATE OF ROTATION OF MIXER DRUM ON STRENGTH OF CONCRETE.

Series 93—28-day tests similar to Fig. 40.

1. *General Discussion of Time of Mixing.*—Table 12 gives the relative strengths of concrete for mixing periods varying from 15 sec. to 10 min. for representative ranges of the variables. The values are based on the general trend of values from these tests. The percentages are referred to the strength of the same concrete mixed 1 min. This period was chosen for comparison for the reason that 1 min. is generally specified as a minimum mixing time for first-class concrete such as that used in road and building construction.

The effect of time of mixing on the strength of concrete is shown graphically in many of the diagrams. It is seen that the strength increases with increased time of mixing in every instance. In all of the tests the strength increases rapidly for the first few seconds of mixing with a gradual reduction in the rate of increase. The relation between strength and mixing-time gives a smooth curve if the values are platted to direct scale. The tests show a consistent relation in all cases between the time of mixing and the other factors. This relation can be expressed by an equation of the form,

$$S = k + n \log t \ldots\ldots\ldots\ldots (6)$$

Where S = Compressive strength of the concrete,
t = Time of mixing.

k and n are constants, whose values depend on mix, consistency, grading of aggregate, age and other conditions of the tests. If this equation is platted using the logarithm of the time and a direct scale of strength we secure a straight line. See Figs. 6, 22, 24, 26, 28 and 30. Due to an error in the diagrams, the abscissæ of the curves are platted in minutes, and the equations expressed in seconds. The use of seconds is preferable, since it avoids the use of negative logarithms.

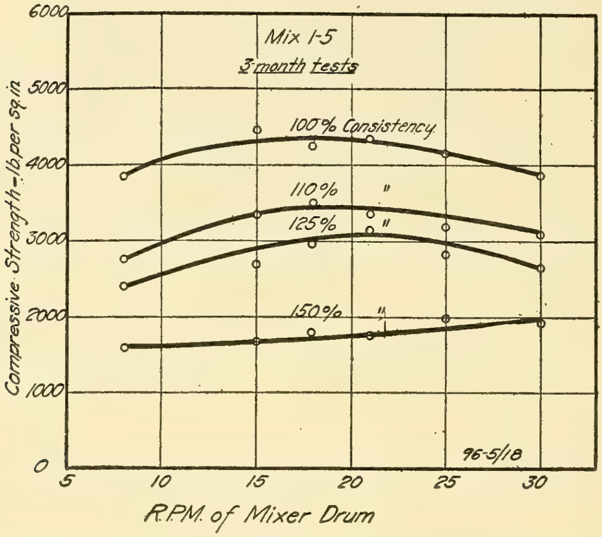

FIG. 42.—EFFECT OF RATE OF ROTATION OF MIXER DRUM ON STRENGTH OF CONCRETE.

Series 93—3-month tests similar to Fig. 40 and 41.

By means of this equation we are able to estimate the relative change in strength which will be produced by a certain change in the mixing period. For the condition of these tests the constants in equation (6) become $k = 900$, $n = 750$. This assumes a 1–4 mix, relative consistency 110 per cent., aggregate graded 0–1¼ in., age at test 28 days. If we wish to compare the strength of the concrete for

1½-min. mixing with that mixed 1 min. we shall have for 1 min. (60 sec.):

$S = 900 + 750 \text{ log. } 60 = 900 + 750 \times 1.778 = 2230$ .... (7)

for 1½ min. (90 sec.) we shall have,

$S = 900 + 750 \text{ log. } 90 = 900 + 750 \times 1.954 = 2360$ lb. per sq. in... (8)

In other words the strength after 1½-min. mixing is 6 per cent. greater than at 1 min. Theoretically equation (6) says, using the constants found in (7) and (8), that for the conditions of these tests we have a strength of 900 lb. per sq. in. at a mixing time of 1 sec. (log. $1 = 0$) and that the strength increases 750 lb. per sq. in. for each succeeding number of seconds corresponding to a logarithmic increase of unity. Thus for

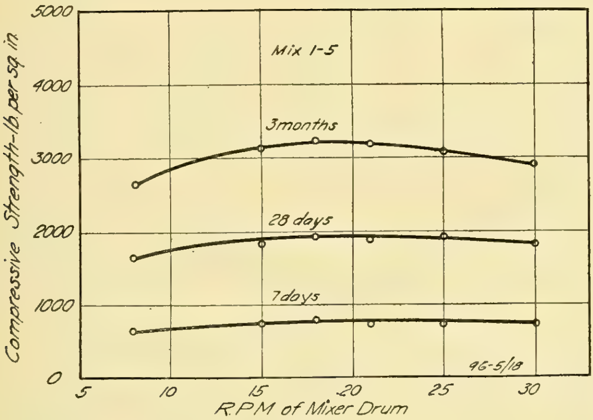

FIG. 43.—EFFECT OF RATE OF ROTATION OF MIXER DRUM ON STRENGTH OF CONCRETE.

Series 96—Each curve is the average of all consistencies for a given rate.

10 sec. (log. $10 = 1$), $S = 900 + 750 = 1650$ lb. per sq. in.
100 sec. (log. $100 = 2$), $S = 900 + (2)\ 750 = 2400$ lb. per sq. in.

In other words there is the same increase in pounds per sq. in. from 1 to 10 sec., from 10 to 100 sec., and from 100 to 1000 sec. A similar interpretation can be placed on other equations of this form.

This equation expresses the law of change in strength for different mixing periods. It indicates the true relation up to the maximum period covered by these tests, that is, 10 min. The maximum time to which this relation may hold was not determined, but it is obvious that it cannot be continued beyond the period at which setting of the cement begins. An equation of this form is useful in making it possible to calculate the effect of mixing for periods of time not covered by the tests.

The values in Table 12 (p. 80) show what may be expected from varying the time of mixing of concrete of different consistencies, mixes, etc. The strengths have been reduced to a percentage of the strength of the same concrete at 1 min. mixing. These values are expressed by the equation:

$$P = l + m \log t \quad \text{...............} (9)$$

Where P = percentage of strength as compared with the 1-min. strength of the same concrete.

t = mixing time in seconds.

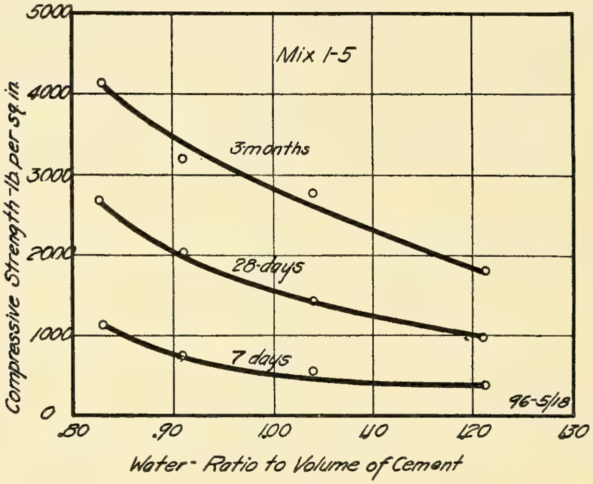

FIG. 44.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.
Series 96—Each point is the average of all rates of rotation for a given consistency.
Compare Fig. 7, 9, 31, 33, 52.

l and m are constants, depending upon the same factors as k and n in equation (6). Equation (9) is, of course, only another way of writing equation (6). The values are based on the general trend of the results found in these tests. It should be borne in mind that these values do not necessarily represent conditions for mixers of other types or sizes.

2. *Concrete of Different Consistencies.*—The tests in Series 89 show the effect of time of mixing when concrete of different consistencies is used. The relative values in Table 12 show that the drier mixes are more affected by continued mixing than the wetter ones; for instance from ½ to 2-min. mixing we find an increase from 89 to

112 per cent. for 90 per cent. consistency, and from 98 to 105 per cent. for 200 per cent. consistency. For the usual range in consistencies which should be aimed at in most work, say, 100 to 125 per cent., changing the mixing time for a 1–4 mix of ordinary aggregates from ¾ to 1½ or from 1 to 2 min. causes an increase in the strength of about 10 per cent.

From the values in Table 12 it will be seen that mixing a concrete of 125 per cent. consistency for 10 min. gives about the same strength as mixing 110 per cent. for 1 min.; the use of 2 to 3 pints of water more than necessary in a 1-bag batch of concrete is sufficient to counteract the beneficial effects of increasing the mixing time from ¾ to 1½ min. or from 1 to 2 min. No reasonable increase in the period of mixing concrete will compensate for the use of an excess of water in the batch.

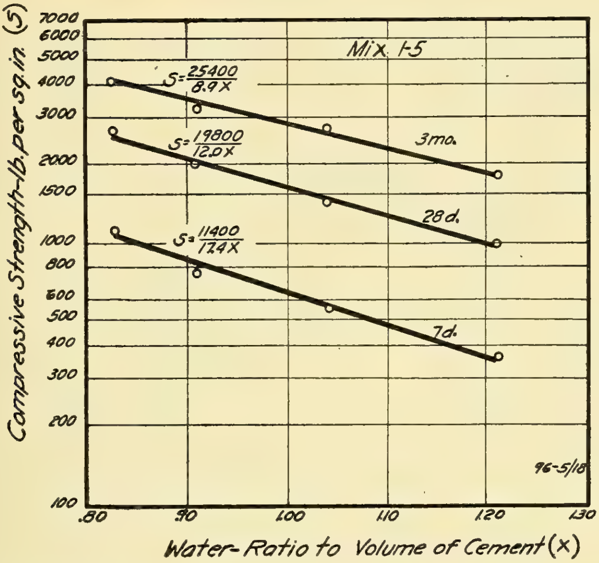

FIG. 45.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.
Series 96—Same data as in Fig. 44, except strength is platted to logarithmic scale.
Compare Fig. 8, 10, 32, 34, 53.

3. *Concrete with Different Quantities of Cement.*—See Series 93 (Fig. 21 to 24) and Table 12. The lean mixes show a much greater relative increase in strength with continued mixing than do the rich mixes. The mixes tested varied from 1–15 to 1–2. For mixing periods from ¾ to 1½ min. or from 1 to 2 min. the increase in strength at 28 days was about 19 per cent. for the leanest mixes and 7 per cent. for

the richest. These percentages are based on the strength at 1-min. mixing. These percentages are not sensibly different for ages of 7 days or 3 months.

4. *Concrete with Aggregates of Different Sizes.*—See Series 93 (Fig. 25–28) and Table 12. Concrete with aggregates of small sizes is more affected by continued mixing than similar concrete with aggregates graded up to coarser sizes. The sizes tested ranged from 0–14 mesh to 0–2 in. For instance, from ½ to 2 min. we find an increase from 64 to 136 per cent. for 0–14 aggregates, and from 91 to 109 per cent. for 0–2 in. aggregates. While the concrete made of fine aggregates increases in strength more rapidly with time of mixing than the coarse, it is not feasible to compensate very great changes in the size and grading of the aggregate by additional mixing. For instance, in order to obtain the same strength as given for a 1-min. mix of 0–1¼ in. aggregate, we would have to mix 0–¾ in. aggregate of the same mix for 10 min. This, of course, is only an indirect

TABLE 12.—EFFECT OF TIME OF MIXING ON THE STRENGTH OF CONCRETE.

Based on 28-day compression tests of 6 by 12-in. cylinders.
The values given in the table are percentages of the strength of the same concrete mixed for 1 min.
These values may be expressed by an equation of the form,

$$P = l + m \log t$$

When P = percentage of 1-min. mix
t = time of mixing in seconds
l and m are constants, which depend on the mix, consistency, age and other conditions of the test.
This equation is only another form of (6) above.

The percentages given represent the general trend of the values, but are not necessarily the exact values shown by the tests for a given time of mixing.

Mix.	Series No.	Relative Consistency, per cent.	Size of Aggregate.	Strength at Different Mixing Periods as Compared with the Strength at 1 min. (P)								Compressive Strength at 28 days, 1-min. mix, lb. per sq. in.	Values of Constants in Equation.	
				¼ m. 15 s.	½ m. 30 s.	¾ m. 45 s.	1 m. 60 s.	1½m. 90 s.	2 m. 120 s.	5 m. 300 s.	10 m. 600 s.		(l)	(m)
1:4	89	90	0–1¼	77	89	95	100	106	112	127	138	3200	+32	39
1:4	89	100	0–1¼	76	88	95	100	107	112	128	141	2650	+28	40
1:4	89	110	0–1¼	81	91	97	100	106	110	123	133	2230	+43	32
1:4	89	125	0–1¼	87	93	97	100	104	106	115	122	1750	+61	22
1:4	89	150	0–1¼	86	93	97	100	104	107	117	124	1000	+58	24
1:4	89	200	0–1¼	90	95	98	100	103	105	112	118	560	+69	17
1:2	93	110	0–1¼	86	93	97	100	104	107	117	124	3450	+57	24
1:3	93	110	0–1¼	76	88	96	100	107	113	129	131	2700	+29	40
1:4	93	110	0–1¼	81	91	97	100	106	110	123	133	2230	+43	32
1:5	93	110	0–1¼	78	89	96	100	107	112	127	138	1800	+33	38
1:6	93	110	0–1¼	78	89	96	100	107	112	127	138	1550	+33	38
1:7	93	110	0–1¼	76	88	95	100	107	113	129	141	1300	+28	41
1:9	93	110	0–1¼	79	89	96	100	106	110	124	135	900	+38	35
1:15	93	110	0–1¼	62	81	92	100	111	119	144	163	350	−12	63
1:5	93	110	0–14	28	64	85	100	120	136	184	222	250	−116	122
1:5	93	110	0–8	39	70	89	100	120	133	176	208	340	−87	107
1:5	93	110	0–4	50	75	90	100	114	125	159	184	440	−49	100
1:5	93	110	0–⅜	79	90	96	100	106	110	125	135	840	+38	35
1:5	93	110	0–¾	73	87	94	100	108	114	132	146	1200	+20	45
1:5	93	110	0–1¼	78	89	96	100	107	112	127	138	1800	+33	38
1:5	93	110	0–2	81	91	96	100	106	109	122	131	1900	+44	32

influence of the water ratio occasioned by differences in size and grading of the aggregate.

5. *Further Remarks on Time of Mixing.*—If the evidence of these tests is a satisfactory guide, it appears that there has been a tendency to overestimate the beneficial effects of increased mixing. It is shown that with this mixer the increase in strength which accompanied increases in mixing time from ¾ to 1½ min., or from 1 to 2 min. for usual concrete is only about 10 per cent. of the 1-min. strength. It seems doubtful if the increased cost of mixing each batch ¾ to 1 min. longer is justified by an increase of 10 per cent. in strength. It is the writer's belief that if concrete is mixed one full minute after all materials are in the drum, we are getting about the maximum efficiency, if the output of the mixer is the factor which controls the amount of work done. Unfortunately, many contractors interpret a 1-min. mix requirement in the specification as meaning 60 seconds if materials happen to be moving slowly, and 10 to 30 seconds when materials are being delivered as intended.

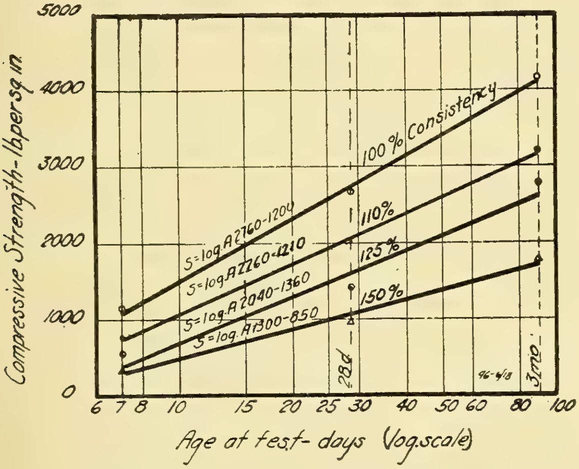

FIG. 46.—EFFECT OF AGE ON THE STRENGTH OF CONCRETE.

Series 96—Each point is the average of all rates of rotation for each consistency. Age platted to logarithmic scale.

Compare Fig. 20, 36, 37, 39.

It is folly to insist on a few seconds increase in mixing time beyond the 1-min. period until more attention has been given to reducing the mixing water within proper limits, especially since we have seen that the wet mixes show decidedly less benefit from continued mixing than the drier ones. A statement made above will bear repetition, namely: No reasonable increase in the mixing time will compensate for an excess of water in the batch.

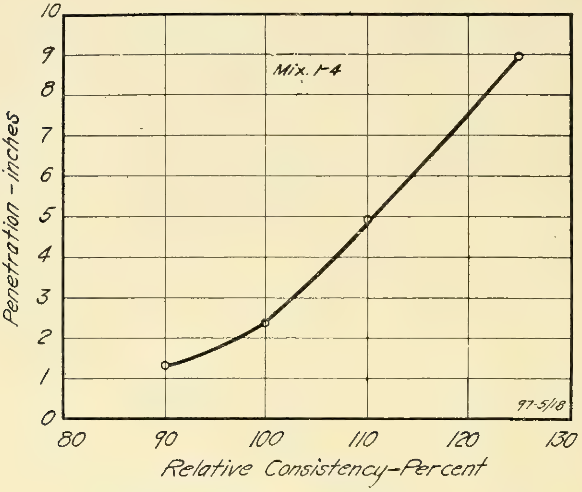

FIG. 47.—PENETRATION TEST FOR CONSISTENCY OF CONCRETE.
Series 96.

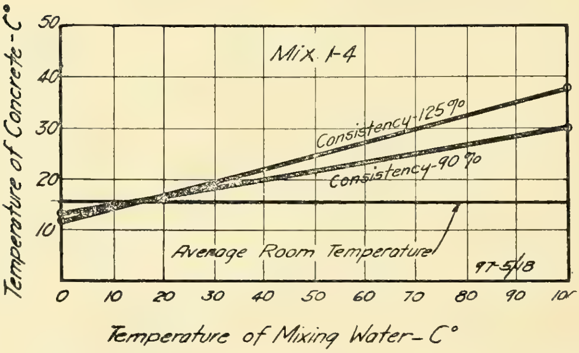

FIG. 48.—TEMPERATURE OF CONCRETE PRODUCED BY MIXING WATER AT DIFFERENT TEMPERATURES.
Series 97—Consistencies between 90 and 125 per cent are omitted.

WEAR TESTS OF CONCRETE.

Wear tests of concrete were made at the age of 2 mo. in Series 89 on a 1–4 mix, aggregates consisting of gravel graded 0–1¼ in. This mix conforms closely to that generally used in concrete road construction. The blocks were stored in damp sand until two days preceding the test, when they were removed to the open room and permitted to dry out. This was done in order that the loss in weight of the blocks due to evaporation might not influence the indications. The wear of the concrete was determined on the basis of the loss in weight during the test. The loss in weight was reduced to the equivalent depth of wear. This seems to be a better measure of the wear, since the loss expressed as a percentage of the weight would be greatly influenced by the original thickness of the block, the unit weight of the aggregates, etc.

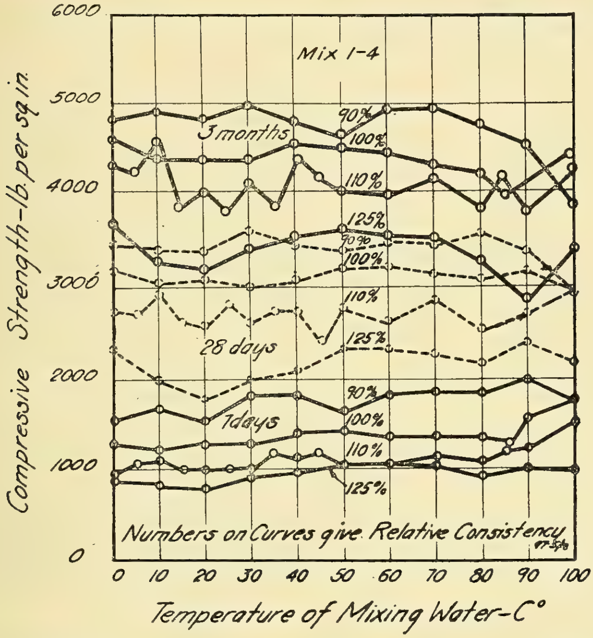

FIG. 49.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE.

Series 97—Each value is the average of 10 tests; 5 from each of 2 batches mixed on different days.

In Fig. 11 the influence of the time of mixing on the wear of concrete is shown for the average of 6 consistencies. The wear increases very rapidly as the mixing period is reduced below about 1 min. For mixing periods between 2 and 10 min. there was practically no variation in the wear. The average wear ranged from 0.65 in. for the longer periods of mixing to 0.9 in. for 15-sec. mixing. It should be borne in mind that in Fig. 11 the 6 consistencies were averaged.

Fig. 12 shows the influence of water-ratio of the concrete on the wear. Here the tests at different times of mixing are averaged. The average wear in this case ranges from 0.55 to 1.1 in., being highest for the wet mixes. The influence of water on wear is approximately the reverse of that found for the strength. Fig. 13 was platted from the curve in Fig. 12. The influence of water content on the wear can be expressed by an equation of the form,

$$W = ab^x \ldots\ldots\ldots\ldots\ldots (10)$$

Where $W =$ depth of wear in inches,
$x =$ water ratio.

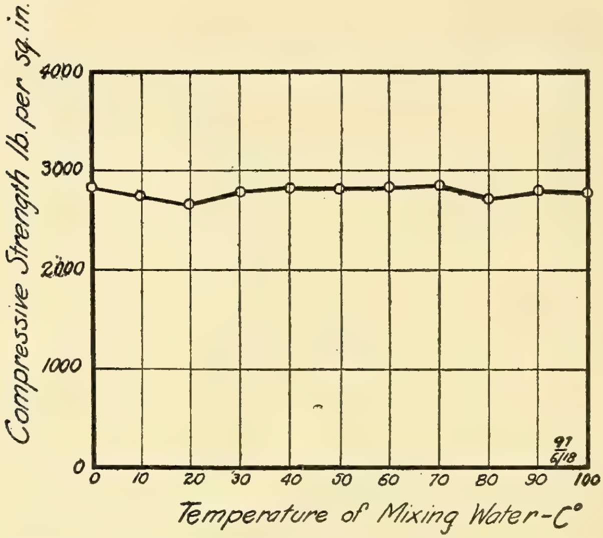

FIG. 50.—EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE.

Series 97—Each value is the average of 120 compression tests of 6 by 12-in. cylinders. This curve is the average of the 12 curves in Fig. 49.

a and b are constants whose values depend on age and other conditions of the test.

It is seen that this relation is of the same form as the equation for strength, except that the exponent which expresses the water ratio now appears in the numerator instead of in the denominator; in other words, increase in the ratio (within the range of plastic mixes) increases the wear and decreases the strength.

For the conditions of these tests equation (10) becomes,

$$W = 0.21\ (4.3)^x \ldots\ldots\ldots\ldots (11)$$

Small changes in the water ratio exert an important influence on the wear. The wear increases at a more and more pronounced rate as the water is increased.

We have sufficient data for a study of the relation between the wear and compressive strength of concrete for a given age. This relation is shown in Fig. 14. Here each consistency is platted separately. The results of an earlier series of tests made in the same rattler on 1–5 hand-mixed concrete, at the age of 4 mo. from Series 75 (details not given in this report) are shown in Fig. 16. The relation between strength and wear of concrete may be found by considering equations (1) and (10) simultaneously. If this is done and the values of the constants found in these series substituted in the equation we shall have,

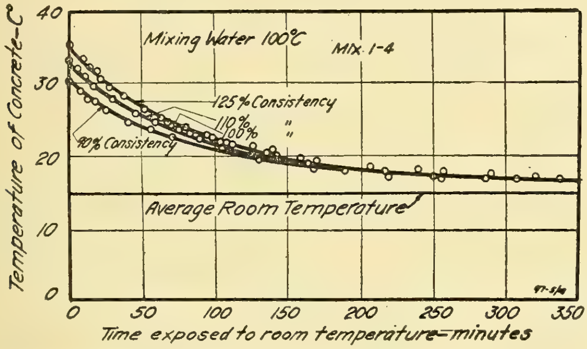

FIG. 51.—TEMPEPATURE OF CONCRETE IMMEDIATELY AFTER MOLDING SPECIMEN.

Series 97—A similar set of curves may be constructed for other temperatures of mixing water.

$$\text{For Series 89:} \quad S = \frac{1800}{W^{1.3}} \quad \ldots\ldots\ldots\ldots\ldots (12)$$

$$\text{For Series 75:} \quad S = \frac{2230}{W^{1.07}} \quad \ldots\ldots\ldots\ldots\ldots (13)$$

Where S = compressive strength in lb. per sq. in.,
W = depth of wear in inches.

These relations are shown in Fig. 15 and 17. It will be noted that this form of equation gives a straight line if both variables (S and W) are platted to logarithmic scales.

These equations are most useful in estimating the probable wearing resistance of a concrete of known strength. Tests now completed do not enable us to analyze the factors which control the values of the constants in equations (12) and (13). It will be of great interest to determine how these terms are affected by such factors as age of concrete, character of aggregate, and other factors. A series of tests is now under way which will throw considerable light on certain phases of this problem.

The interrelation of wear, strength and water content may be shown on one diagram, as in Fig. 18. This diagram is made up by superimposing Figs. 13 and 16 with certain modifications in position. The dotted line with arrows indicates the method of solving the following problem: What wear will be given by a concrete having a compressive strength of 2800 lb. per sq. in.? What is the water ratio for this concrete? The diagram shows a wear of 0.72 in. and water ratio of 0.84. A great variety of other problems may be solved in this way, as: What compressive strength and wear will be given by concrete of a given water ratio? Certain features of these tests suggest that the relation between strength and wear indicated in equation (10) is independent of the age and other properties of the concrete. This, however, cannot now be stated as a definite conclusion.

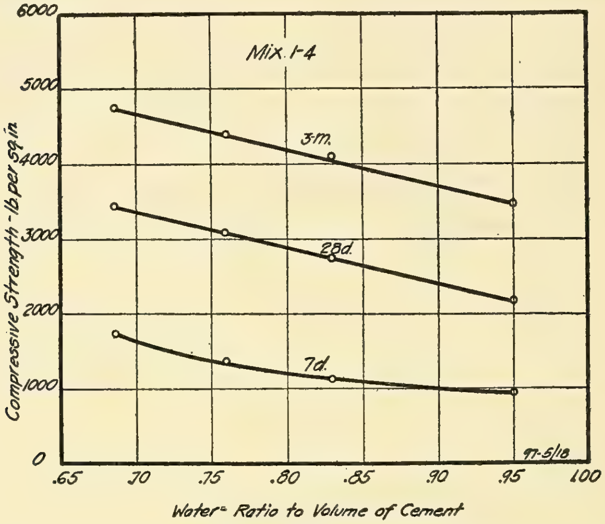

FIG. 52.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.

Series 97—Each value is the average of 110 tests; 10 tests for each of 11 different temperatures. These curves approximate straight lines due to the narrow range in the water content.

Compare Fig. 7, 9, 31, 33, 44.

The Talbot-Jones rattler has proven entirely successful as a method of studying the wear of concrete. The abrasive charge, the rate and number of revolutions used in these tests have been found to give a wearing action suited for a wide range in the properties of concrete. A very poor concrete may be entirely destroyed, a high-grade concrete will show a wear of ½ in. or less.

COMPARISON OF HAND AND MACHINE MIXING

In Series 93 parallel tests of hand-mixed concrete were made for comparison with the machine-mixed specimens. The two sets of tests were made from the same materials under the same conditions. The hand-mixed specimens were mixed separately; the four cylinders in a set were made on different days. It has already been noted that the strength is influenced by the water content in the same way for both machine-mixed and hand-mixed concrete. The tests show that in general the method of hand-mixing, by means of a bricklayer's trowel which has been used in most of our research work, gives results sensibly equivalent to that found by mixing concrete in the mixer for a period of 1½ min. The concrete made by using fine sand for aggregate gave a strength for hand-mixing more nearly equivalent to 2 to 5 min. machine-mixing. Whether this indicates a relative excess in the hand-mixing for the finer sands, or a difference in the performance of the mixer, it is impossible to state; probably both of these factors affect the result.

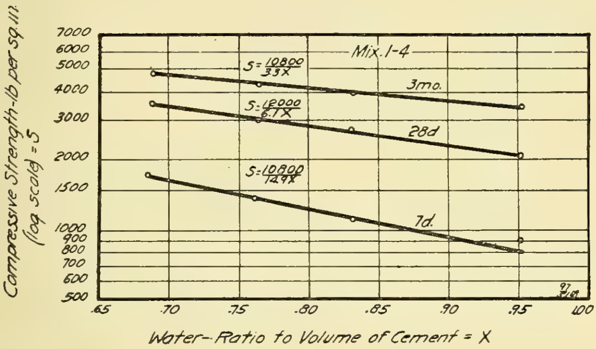

FIG. 53.—INFLUENCE OF WATER ON THE STRENGTH OF CONCRETE.
Series 97—Same data as in Fig. 52, except strength is platted to logarithmic scale.
Compare Fig. 8, 10, 32, 34.

The mean errors of the tests show about the same variation in 4 machine-mixed specimens from the same batch as for 4 hand-mixed specimens made from separate batches on different days. The average mean errors for the 14 sets of 28-day tests in Series 93 are:

Machine	10.4 per cent.
Hand	11.9 per cent.

The mean errors are greatly reduced if a larger number of specimens are made for a given condition. In most of our experimental work 5 specimens constitute a set; in some instances this number is increased to 10 or even 20.

EFFECT OF RATE OF ROTATION OF MIXER DRUM.

In Series 96 tests were made on 1-5 mix, aggregates graded 0–1¼ in. for four different consistencies in which the rate of rotation of the drum was varied from 8 to 30 r. p. m. Tests have been completed in this series at ages of 7 and 28 days and 3 months. The 1-year tests are not yet due. The results of these tests are given in graphical form in Fig. 31 to 46. In Fig. 43 all consistencies have been averaged for each age. It is seen that the rate of rotation of the mixer drum exerts only a slight influence on the strength of the concrete for rates between about 12 to 25 r. p. m. For the low and the very high rates there is generally a slight reduction in strength. The rate of 18 r. p. m. recommended by the manufacturer for this machine corresponds closely to the rate for maximum strength. This rate gives a peripheral speed of 150 ft. per min. for the maximum diameter of the inside of the drum.

In studying these series of tests particular weight should be given to the 110 per cent. consistency, since these tests are the average of three batches for each rate mixed on different days. However, there seems to be little difference in the behavior of concretes of different consistencies with reference to the effect of the rate of rotation.

EFFECT OF AGE ON THE STRENGTH OF CONCRETE.

In Series 89, 93, 96 and 97 compression tests were made at ages of 7 days, 28 days and 2 or 3 months. Other tests will be made in Series 93 and 96 at the age of 1 year. The data in the tables enable us to make a study of the increase in strength with age. It should be borne in mind that all the specimens were stored in damp sand until tested.

Fig. 19, 21, 35 and 38 show the relation of age to strength platted to a direct scale of ages in days. Corresponding diagrams are given in several instances in which the age is platted to a logarithmic scale. form

$$S = C \log A - D \quad \dots\dots\dots\dots\dots\dots (14)$$

Where S = compressive strength

A = age at test.

C and D are constants which depend on mix, consistency, time of mixing and other conditions of the test. For concrete of usual quality as used in these tests this equation becomes

$$S = 2200 \log A - 1200 \quad \dots\dots\dots\dots (15)$$

Where the age at test (A) is expressed in days.

Equation (15) is based on tests of 1–4 and 1–5 mixes, 110 per cent. consistency, mixed 1 min.

An equation of a somewhat similar form is used by Unwin* for expressing the effect of age on the tensile strength of cement and mortars, as follows:

$$y = a + bx^n \quad \text{.....................} \quad (16)$$

Where y = strength of cement or mortar,
x = age at test,
n = an exponent,
a and b are constants,

for neat cement he found

$$y = 363 + 48\sqrt[3]{x-1} \quad \text{............} \quad (17)$$

Where x = age in weeks.

Equation (15) seems to represent more nearly the relation for concrete tests. This equation, also, is of the same type as those found in other relations. Compare equation (6).

The important thing is that at no time is there any falling off in the strength of the concrete. The erroneous opinion held by many that concrete decreases in strength is based on the results of briquet tests of cements and mortars. This reveals a fundamental objection to the briquet. A retrogression in strength of concrete has not been found in any group of tests made in this Laboratory. Hundreds of groups have been tested at ages from 7 days to 1 year or older. This increase in strength persists regardless of mix, size of aggregate, consistency of concrete, time of mixing, or other variables. Reports of published tests show that a similar relation exists up to periods of 9 years. It is not known how long this increase in strength continues, but there is no apparent reason why it should cease, so long as the concrete does not dry out, and we do not exceed the strength of the aggregate.

According to equation (15) we should expect the following strengths for concrete of the quality used in these tests at the ages shown:

1 Week	1 Month	3 Months	1 Year	2 Years	5 Years	10 Years
660	2050	3100	4440	5100	5900	6600

Using the 1-mo. strength as a basis of comparison, we obtain the following percentages:

Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.	Per Cent.
32	100	183	240	250	288	322

"The relatively low strengths shown at the earlier ages is probably due to the fact that the tests were made in mid-winter.

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*See "Testing of Materials of Construction," by W. C. Unwin, 1910, p. 459.

The specimens were stored in the basement where the temperature was somewhat low. Tests made under normal atmospheric conditions show a strength at 7 days of from 40% to 50% of the 28-day strengths."

There is no further increase in strength after the concrete has thoroughly dried out. The fact that increase in strength continues so long as the concrete is kept damp, and ceases as soon as it becomes dry, emphasizes the necessity for protecting concrete from too early drying and for continuing the curing in a damp condition as long as possible.

Since the strength is a logarithmic function of the age, it is clear that a correct distribution of ages for testing purposes would be such as would give a constant difference in their logarithms. This condition is approximately fulfilled in the ages generally used in these tests; that is, 7 and 28 days, 3 months and 1 year. The difference in this case varies from log. 3 to log. 4. This means simply that when age is platted to a logarithmic scale, the points are uniformly distributed along the curve. The next period in regular sequence would be 3 to 4 years.

It is interesting to note that the increase in strength with age (if the concrete is damp) is analogous to that found by increasing the time of mixing, increasing the quantity of cement, or using coarser aggregate. The last two items, of course, may be combined in the one item of reducing the water ratio in the mix. It appears that proper curing is by all odds the cheapest method of getting full value from the cement used.

EFFECT OF TEMPERATURE OF MIXING WATER ON THE STRENGTH OF CONCRETE.

In Series 97 compression tests were made on concrete of 1–4 mix in which the temperature of the mixing water ranged from 0 to 100°C. Duplicate batches were mixed on different days using four different consistencies. Fig. 49 shows the average strengths obtained at 7 and 28 days and 3 months. An average curve for all consistencies and all ages is shown in Fig. 50. It may be surprising to find that such a wide range in temperature should make no more change in strength. The reason for the uniformity in strength is found in Fig. 48 and 51. Fig. 48 shows how the temperature of the concrete is influenced by the temperature of mixing water. The temperature of the concrete was determined by inserting a thermometer in the fresh concrete immediately after molding the specimens. For average consistency the temperature of mixing water must be changed about 5° in order to produce 1° change in the temperature of the fresh concrete. There is only a small difference for wide range in consistencies. Fig. 51 shows that, regardless of the consistency, the concrete rapidly approaches room temperature. The most favorable condition is platted—100°C. mixing water. The concrete probably reaches room temperature before initial setting of the cement occurs.

It is important to note that the mixing water near the boiling point has no injurious effect. It has been reported recently that certain concrete was defective on account of being "cooked" by using boiling water for mixing.

In large masses of concrete the use of hot water for mixing will exert a greater influence on the temperature. In freezing weather hot water is advantageous in eliminating danger from frozen aggregates.

YIELD AND DENSITY OF CONCRETE.

In all the tables values of "density" and "yield" of concrete are given. The yield is based on the volume of aggregate as used; for example, a yield of 1.025 means that the volume of concrete is 2½ per cent. greater than the volume of the mixed aggregate. The method of determining the unit weights of aggregates described above gives volumes of aggregates which correspond closely to the actual quantities of materials in a concrete of the usual mixes. The yield is only slightly affected by time of mixing; the longer mixing times giving slightly lower values than the short.

The term density is used to express the total volume of solid material in the fresh concrete. It is based on the specific gravities of the cement and aggregate, the proportions of the materials and the yield of the concrete. The density is only slightly affected by time of mixing of the concrete; it is more influenced by variations in consistency, mix, grading of aggregate, etc. There is in general no definite relation between strength of concrete and density. The relation is sometimes direct and sometimes the reverse if we consider all possible proportions of given materials. The maximum density for these materials was found in the 1:7 and 1:9 mixes. It will be noted (Table 8) that the rich mixes give lower densities than the leaner ones. With a systematic variation in some one factor in either the proportions of given materials or in the manipulation of the concrete, we generally find a systematic relation between strength and density. Wide variations in strength of concrete are accompanied by slight changes in density. Density can be determined only after the concrete is mixed and necessitates many careful weighings and measurements, as well as much laborious computation. The water ratio gives a much more usable function for judging of the relative strength of concrete of any proportion. No special determinations or calculations are necessary. The only requirement is that the quantities of water and cement be known. The relative strength of concrete can be determined before mixing. Other tests have shown that this method is applicable to any aggregate if the quantity of water absorbed is taken into account.

The reader is cautioned against attempting to draw conclusions from these tests with reference to the use of the density of the concrete as a method of proportioning aggregates, since no variation was made in the character of sieve analysis curves. Other tests have shown that wide variations in density may occur due to variations in the grading of the aggregate without any change in strength.

DISCUSSION.

Mr. Aldrich. MR. R. C. ALDRICH.—Were any experiments made with reference to different methods of mixing, that is, putting in the water first and then the cement, or putting in some of the aggregate first. It seems to me these would be very valuable. We have had reference to that in two of the papers in this meeting already, where it has been said that better results were obtained by putting in the water first, then the cement, and, in one case, fine aggregate was put in and then coarse aggregate.

Mr. Abrams. MR. D. A. ABRAMS.—We have not overlooked that fact, although no tests have been made to bring that out definitely. All the material was put in the drum before the mixer was started. The time of the mixing was counted from the time the drum was started until the time of beginning the discharge. You can readily see that for very short times of mixing, it makes a good deal of difference as to just when you start. This problem just mentioned is not quite so simple as it might seem at first thought. I hear claims for varying the sequence of materials made frequently by people who use different methods of charging the mixer, but I must confess that I see no logical reason for that being true. The conclusions seem to be based on general impressions and not on very definite information. It is a good field for study and we shall find it desirable to make such investigation as soon as feasible.

Mr. Hatt. MR. W. K. HATT.—The author referred to the tests of wood by the Forest Service. For a long term of years, the science of wood technology went through somewhat the same progress as the science of mixing concrete apparently is going through now. A number of unrelated tests of wood, out of which no general conception or correct viewpoint could be obtained, were made. It was only after some fundamental studies to ascertain the natural laws that are operative were completed that we were able to get a proper viewpoint of the strength of wood and the condition it is in the industries so as to produce the best product.

I think that such studies as are reported to this convention will lead you to somewhat the simple viewpoint of the mechanism which operates in the mixing and hardening of concrete, and then the further light will come on the operation of mixing concrete. Therefore, such fundamental studies as these are of the utmost importance.

I have said before, that there is a marked similarity between the laws which control wood and those which control concrete, as if there were some fundamental condition that was common to both. For instance, wood has plasticity; concrete has plasticity in the long-continued loads. Wood is very markedly affected by moisture conditions; that is, moisture is taken on from the air. Concrete exhibits the same property. There is evidence to show that concrete behaves quite differently under time rate of application of load. Wood has the same factor. Some two or three years ago our cement

Mr. Hatt. experts used to tell us about colloids in the cement. We know that wood has a colloidal structure. If concrete has a colloidal structure, there is no reason to suppose that the same laws would not control both.

Mr. Slater. Mr. W. A. Slater.—While the question of the effect of water on concrete is up, it seems as though it might be of interest to call attention to one feature of the paper on the Western Newspaper Union Building test. Prof. Talbot reported that the effect of the standing of the load on the floor was very slight. This building was approximately 8 years old, and undoubtedly the concrete must have been very thoroughly seasoned and very dry. The amount of water must have a great deal to do with the plasticity, and I believe the same thing has been pointed out as being true with reference to wood. I might point out in addition, that the same thing is true with reference to gypsum. Gypsum is coming to be used somewhat as a building material and some studies have been made of its properties. We find that practically identical relations hold with gypsum as those that have been pointed out here with concrete, in relation to the amount of water contained. The strength decreases just as the strength of concrete decreases with increase in the amount of water, and the plasticity increases with the increase in the amount of water present in gypsum. The longer the specimen dries the straighter is the stress strain relation, until, with the thoroughly dry specimen of gypsum we have almost as straight a stress-strain relation as for a steel specimen. It has no direct bearing perhaps on the question of concrete, but it is a similar relation to those that have been pointed out, and it may be worth while to call attention to it.

TESTS OF BLAST FURNACE SLAG AS THE COARSE AGGREGATE IN CONCRETE.

By P. J. Freeman.*

Blast furnace slag has been used as the coarse aggregate in concrete for a number of years, and numerous laboratory tests have been made during that time; but these tests were conducted by different individuals, and with different cements and fine aggregates, and it is difficult to make direct comparisons of the results obtained.

In order to secure comparative data, a series of tests was planned in which the materials would be collected from various localities at one laboratory, and where all specimens would be made by the same operators, under uniform conditions.

The whole 5-yr. series of tests will not be completed until December, 1921, and this paper only includes the data obtained during the first year.

Purpose of Tests.

The purpose of this series of tests was to furnish information relative to the use of concrete materials, as follows:

(1) A comparison of the crushing strengths of air-cooled blast-furnace slag, limestone, granite, trap rock and gravel, when used as the coarse aggregate in concrete; tests to be made at the end of 14, 30, 60 and 180 days, 1 year, 2 years, 3 years, 4 years and 5 years.

(2) Determination of the corrosive tendency of sulphur in slag.

(3) Effect of sulphur and other elements on the durability of concrete up to the age of 5 years.

(4) Relative strength and durability of concrete made of high magnesia, low lime slag, and low magnesia high lime slag.

Materials Used.

The materials used as the coarse aggregates in these tests were secured from the following localities:

		Mark.
Slag	Cleveland, Ohio	87410
Slag	Duquesne, Pa.	87420
Slag	Youngstown, Ohio	87430
Slag	Mayville, Wis.	87440
Slag	Sharpsville, Pa.	87450
Slag	Cleveland, Ohio	87470
Slag	Ensley, Ala.	87480
Slag	Pottstown, Pa.	87520
Slag	Toledo, Ohio	87530

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* Engineer of Tests, Pittsburgh Testing Laboratory, Pittsburgh Pa.

		MARK.
Gravel	Pittsburgh, Pa	87460
Trap rock	Birdsboro, Pa	87490
Gravel	Akron, Ohio	87500
Crushed granite	Stockbridge, Ga	87510
Limestone	Gates City, Ala	87540
Limestone	Cleveland, Ohio	87550

It did not seem practicable to screen the fine aggregate, and recombine, to conform to Fuller's curve, or to use a combination of two or more sands, which would make theoretically the best fine aggregate. The material selected was reasonably well graded and the same sand was used throughout the series of tests, the whole amount being secured at one time, from the back channel of the Ohio River, at Neville Island.

CHEMICAL ANALYSES OF SLAG.

	CEMENT LABORATORY NUMBERS.								
	87410	87420	87430	87440	87450	87470	87480	87520	87530
Silica	35.35	36.72	34.35	31.98	35.74	33.63	35.55	31.53	35.15
Alumina	12.28	12.81	13.55	12.25	11.81	13.41	12.04	12.39	14.11
Iron oxide	0.90	1.50	1.02	2.00	0.38	1.25	0.62	0.50	0.25
Manganous oxide	0.50	0.61	0.71	0.34	0.70	0.40	0.36	0.22	0.43
Titanium oxide	0.40	0.33	0.50	0.30	0.30	0.40	0.40	0.55	0.45
Calcium oxide	40.05	40.98	45.80	31.75	42.65	39.18	41.35	32.78	38.75
Magnesium oxide	8.20	5.14	1.62	19.43	6.18	9.31	8.45	18.24	8.84
Sulphur in calcium sulphide	1.05	1.29	1.10	0.89	1.26	1.03	0.73	0.89	1.11
Sulphur in calcium sulphate	0.52	0.27	0.41	0.27	0.24	0.20	0.50	0.10	0.17
Sulphur in other forms	....	0.12	....	0.07	....	....	0.05	0.18	....
Carbon	....	....	....	....	....	....	....	0.62	....

The cement used was Alpha portland. This brand was selected by lot, being drawn from a list of several standard brands of portland cement. All cement was purchased at the same time, and tested and found to be satisfactory before the preparation of concrete test specimens was begun.

SCREENING AND RECOMBINING OF COARSE AGGREGATES.

As the various aggregates were received, they were screened through iron plates with circular holes, of the following diameters: $1\frac{1}{4}$, 1, $\frac{3}{4}$, $\frac{1}{2}$, and $\frac{1}{4}$ in.

The portions retained on each of the above sieves were stored separately, and labeled, to be later recombined to make the coarse aggregate used in the tests.

The coarse aggregate was recombined to conform to Fuller's curve by weighing equal quantities of the four gradings, and shoveling them together, turning them until their appearance showed them to be thoroughly mixed.

In order accurately to proportion the concrete, the weight per cubic foot of all materials was determined. Since there is no generally accepted method for determining the weight per cubic foot of concrete materials, one was used which had been found in the past to give consistent results.

A cubic foot measure was filled loosely with either sand, or the recombined aggregate, after which the measure was dropped ten times on a felt pad 1 in. thick, from a height of 3 in. The measure was again filled and smoothed off with a straight-edge, and weighed. The average of ten determinations was taken as the weight per cubic foot of the material used. The variation of the individual determinations was usually within 0.5 per cent, and seldom over 1 per cent. The weight per cubic foot was frequently redetermined, to take into account any drying out of the material. The weight of the cement per cubic foot was taken at 100 lb., this being a very convenient figure for computations and one frequently used.

PROPORTIONS FOR CONCRETE.

Void determinations were made on the various aggregates, after recombining. Each coarse aggregate was thoroughly wet, drained and a cubic foot measure filled and weighed, as given in the method for determining the weight per cubic foot. Water was then slowly added until the measure was level full. From the increase in weight the percentage of voids was computed.

It was not possible to combine the sand and cement, with the slag, gravel and crushed stone, respectively, to strictly conform to Fuller's curve, and still have tests which would be comparable with each other on the basis of equal proportions of cement. It was, therefore, necessary to determine the leanest mixture which would produce a dense concrete when using the coarse aggregate having the highest percentage of voids, and then using this mixture for all materials. By this method, the same quantity of cement was used to make each specimen, and the test data shows a comparison of the different aggregates under the same condition.

The proportions for the mortar were determined by making trial mortars of various proportions of cement and sand, and selecting the mixture giving the maximum density as shown by increase in volume of the resulting mortar. After numerous tests, the proportions of 1 part cement and 2 parts sand, were found to most nearly fulfil the tests for maximum density of the mortar.

The coarse aggregates used for these tests, varied in weight per cubic foot from 64 to 104.5 lb., and the percentages of voids from a minimum of 31.85 to 49.2 per cent. Since the percentage of voids in one case was 49, to obtain the maximum density, using this aggregate, the mixture should be almost exactly two parts of mortar and four of coarse aggregate, this proportion would give some excess mortar in all of the other cases.

The fact that a 1:2:4 mixture is one which is very commonly used, and a large amount of data may be found for comparison, was an additional reason for using it in these tests.

These proportions by volume, having been selected, the equivalent weight of the materials for these proportions was determined, and throughout the series of tests, all materials were weighed, and greater accuracy in proportioning thus obtained. The mixture, however, is by volume, the method of weighing being used only to insure more accurate proportions.

METHOD OF MIXING SPECIMENS.

A quantity of material, sufficient to make ten cylinders, was mixed at one time, the sand being spread in a flat pile, and the cement placed over this. The two materials were turned by two men until the color appeared to be uniform, three or four turnings being required. The coarse aggregate was then shoveled on this material, and the whole turned dry three times. During the fourth turn a weighed amount of water was added from a sprinkling can, and three additional turnings given the mixture. During the last three turnings small quantities of water were added as needed, until a "quaking consistency" was obtained.

In all mixtures an attempt was made to secure the same consistency regardless of the amount of water used. For this reason, it was not possible to use a mechanical mixer, as the quantity of water is very important, and in mechanical mixing the material may be made too wet, and the whole batch spoiled for laboratory purposes. It is noteworthy that care must be used to obtain the correct consistency, and that the addition of 1 lb. of water to a 10 specimen mixture, would give a consistency too wet, usually described as "mushy," and the results of the tests would be unsatisfactory.

METHOD OF MOLDING SPECIMENS.

The specimens were made in steel molds 8 in. in diameter by 16 in. high. The concrete was poured into these molds in layers 4 in. thick, and each layer tamped, or puddled thirty times with a ⅞-in. round rod. After the second and fourth layers, the sides were spaded with a large trowel. These cylinders were finally finished at the top by spading with a small trowel to form a smooth upper rim, and a piece of plate glass placed on top, to form a smooth surface. Since the concrete would settle slightly after a few hours, it was necessary to cap the top of the specimen with plaster of paris and cement; and again place the plate glass on the cap, to make a smooth surface.

The specimens were kept in the molds for forty-eight hours, and then stored in damp sand for thirty-five days, or until tested. At the end of this time all specimens were removed and stored in air. Four short pieces of reinforcing steel were embedded in each of two cylinders from every batch.

These pieces were 3, 6, 9 and 12 in. long, and were cut from ½-in. twisted reinforcing bars having the following chemical analysis:

	PER CENT.
Carbon	0.20
Manganese	0.45
Phosphorus	0.018
Sulphur	0.046

These specimens will be examined at the end of the five-year period to determine the corrosive action of the aggregates.

Specimens marked 1 to 10 were made at one time as were 11 to 20 and 21 to 30.

The results up to one year are shown in the accompanying large table.

P. T. L. Laboratory No.	Name of Material Used.	Weight per cu. ft. of Material Used.	Per Cent of Voids in Material Used.	14-DAY TESTS.					30-DAY TESTS.					60-DAY TESTS.					180-DAY TESTS.					ONE-YEAR TESTS.				
				Mark on Cylinder.	Weight of Cylinder.		Crushing Strength, lb. per sq. in.	Average Crushing Strength, lb. per sq. in.	Mark on Cylinder.	Weight of Cylinder.		Crushing Strength, lb. per sq. in.	Average Crushing Strength, lb. per sq. in.	Mark on Cylinder.	Weight of Cylinder.		Crushing Strength, lb. per sq. in.	Average Crushing Strength, lb. per sq. in.	Mark on Cylinder.	Weight of Cylinder.		Crushing Strength, lb. per sq. in.	Average Crushing Strength, lb. per sq. in.	Mark on Cylinder.	Weight of Cylinder.		Crushing Strength, lb. per sq. in.	Average Crushing Strength, lb. per sq. in.
					Lb.	Oz.				Lb.	Oz.				Lb.	Oz.				Lb.	Oz.				Lb.	Oz.		
87410	Slag Cleveland, Ohio	66.5	49.2	9 18 29	62 62 63	8 10 3	1897 1998 1928	1941	10 33 28	63 63 62	3 10 14	2461 2770 2343	2525	8 13 26	63 62 62	00 15 13	2815 2966 3008	2930	7 5 8	62 62 63	10 5 0	3740 3958 3560	3753	2 16 7	62 62 62	4 10 10	4300 4774 4420	4498
87420	Slag Duquesne, Pa.	78.5	42.74	9 18 28	65 65 65	10 13 8	2212 2318 1946	2159	10 17 27	66 64 65	00 4 1	2425 2983 2562	2657	4 13 22	65 65 65	6 6 4	3143 3402 2810	3117	7 16 25	64 65 64	12 10 5	4280 4464 4200	4315	21 5 8	65 64 65	00 1 11	4824 4544 4808	4725
87430	Slag Youngstown, Ohio	79.3	43.15	9 18 27	65 64 65	4 10 4	2346 2128 1928	2134	10 17 28	65 64 64	8 00 10	2642 2568 2761	2657	7 12 22	66 65 65	4 10 12	3625 3220 3074	3306	2 16 20	65 65 65	10 00 5	3880 4130 4452	4154	1 5 11	65 65 65	8 10 6	5092 5148 4844	5028
87440	Slag Mayville, Wis.	64.5	45.87	2 12 22	62 62 63	12 12 4	2238 2141 2238	2206	3 13 23	63 62 63	00 12 4	2640 2630 2688	2653	5 21 28	62 63 62	5 00 11	3523 3365 3320	3403	1 4 6	62 62 63	15 8 0	4146 4268 4512	4309	14 8 10	62 62 62	6 2 14	4928 4556 5172	4885
87450	Slag Sharpsville, Pa.	75.0	41.53	2 12 28	65 65 64	14 3 14	2477 2380 2594	2484	3 13 27	65 65 65	00 00 00	3127 2999 3100	3075	10 18 21	65 65 64	11 00 13	3359 3468 3268	3365	1 5 9	64 65 64	11 3 11	4906 4678 4824	4803	6 19 17	65 64 65	2 13 5	5397 5660 5108	5388
87460	Gravel Pittsburgh, Pa.	104.5	31.85	2 12 22	69 68 68	2 13 11	2045 2093 2000	2046	3 13 23	68 68 68	13 9 15	2608 2514 2409	2510	6 21 27	68 69 68	13 2 13	3427 3170 3289	3295	7 9 19	68 68 69	14 9 1	4200 3816 3892	3969	14 11 20	69 69 69	3 2 3	4556 4664 4660	4627
87470	Slag Cleveland, Ohio	64.0	46.77	2 12 22	62 62 61	4 4 11	2387 2237 2146	2257	1 11 21	62 61 62	2 8 5	2810 3057 2666	2844	3 14 24	61 62 61	10 1 2	3167 3126 3068	3288	6 13 16	61 61 61	15 14 9	4588 4422 4172	4394	8 23 4	62 61 62	12 11 1	5152 4872 4844	4956
87480	Slag Ensley, Ala.	83.8	42.00	2 10 21	66 66 66	13 00 10	2043 2160 2126	2109	1 11 20	66 66 67	9 00 2	2660 2800 2796	2752	8 17 25	64 66 66	00 9 5	3270 3354 3244	3289	3 4 7	66 66 66	2 9 7	4432 4460 4460	4451	15 12 14	66 66 65	4 3 11	5388 4748 5180	5105
87490	Trap Rock Birdsboro, Pa.	98.7	41.93	2 12 22	72 73 72	8 1 3	2109 2053 2026	2063	1 11 21	72 73 72	6 5 9	2454 2330 2374	2386	9 15 20	72 72 72	6 12 8	3411 3416 3256	3360	3 6 7	72 72 72	15 12 14	4814 4738 4906	4819	4 26 14	72 73 73	9 3 9	4888 4666 5296	4950
87500	Gravel Akron, Ohio	95.0	35.9	2 12 22	68 68 67	00 7 10	1793 1800 1792	1795	1 11 21	66 67 67	13 9 8	2040 2040 2153	2078	8 16 28	67 68 67	12 2 14	2756 2378 2527	2554	3 4 16	68 68 68	3 9 0	3636 3840 3404	3627	5 9 10	67 66 67	4 13 13	3790 3710 3520	3673
87510	Crushed granite Stockbridge, Ga.	90.0	42.34	11 21 22	69 68 69	00 00 8	1980 2178 2208	2122	1 12 22	69 68 69	00 4 8	2230 2334 2313	2292	3 16 19	69 69 69	2 00 8	3112 2760 3258	3043	8 17 23	69 69 69	6 0 5	4190 4016 4248	4151	6 10 24	69 68 69	00 12 2	4740 4422 4738	4633
87520	Slag Pottstown, Pa.	73.75	43.55	1 12 22	63 64 63	00 00 00	2151 2167 2244	2187	2 11 21	64 63 63	2 6 00	2738 2600 2613	2650	5 14 24	63 63 63	8 00 8	3245 3244 3378	3289	27 4 3	63 63 63	5 6 0	4210 4203 4140	4184	18 10 9	63 63 63	10 8 8	4620 4576 4750	4649
87530	Slag East Toledo, Ohio	81.75	42.5	2 12 21	65 66 66	13 14 10	1918 1856 2051	1942	1 11 22	66 65 66	7 4 4	2527 2402 2680	2536	7 16 20	66 66 66	3 5 4	3251 2864 3195	3103	3 9 13	66 66 65	4 15 6	4130 4333 4030	4164	5 X 4	66 67 65	00 2 14	4972 4794 4596	4787
87540	Limestone Gates City, Ala.	94.87	40.33	1 6 7	69 69 71	8 8 0	1670 1750 1720	1713	3 10 23	69 69 69	9 8 8	1985 1950 2030	1988	2 15 22	69 68 69	6 13 00	3149 3014 3072	3082	4 8 9	70 69 68	0 6 12	3936 4636 3814	4127	3 1 6	70 69 70	00 11 12	4340 4344 4348	4344
87550	Dolomitic Limestone Cleveland, Ohio	94.11	38.71	4 13 23	69 69 69	11 7 0	1830 1814 1769	1804	1 12 24	69 69 69	6 14 00	2269 2442 2375	2360	2 17 22	70 69 70	00 12 12	3503 3846 3462	3604	3 19 27	70 70 69	0 0 0	4640 5011 4520	4724	12 25 20	69 69 69	4 2 12	5030 4874 4808	4904

DISCUSSION OF RESULTS.

(1) Some of the materials required more careful placing in the molds than others, in order to obtain perfect specimens and to insure all specimens being smooth on the outside when the molds were removed, a greater amount of time and labor was employed than would be the case if the specimens were prepared in the field. The crushing strengths were, probably, also made somewhat higher.

Each specimen received exactly the same treatment, and all data should be comparable.

(2) The length of time during which this series of tests has been conducted does not warrant the drawing of any definite conclusions, but a general uniformity of the results of the crushing tests of the concrete may be observed.

(3) It will be noted that about one-half of the tests of the slag concrete were made using slag produced by the quick-cooling process, in pits, in which the slag is shipped within a few days from the time it comes from the furnace, and the remainder from slag which had been seasoned in banks for a period of six months in some cases, and as much as 15 years in one case.

(4) Slags coming from furnaces many hundred miles apart varying quite widely in chemical analyses, and also varying considerably in the weight per cubic foot, do not vary in strength in proportion to either the weight or percentage of any chemical constituent.

SOME TESTS ON THE EFFECT OF AGE AND CONDITION OF STORAGE ON THE COMPRESSIVE STRENGTH OF CONCRETE.

By Harrison F. Gonnerman.*

This paper presents the results of some tests on the development of compressive strength of concrete with age and on the effect of condition of storage on the compressive strength. The results show little increase in strength in the air-stored specimens after an early age and a marked increase in strength in the specimens stored under moist conditions up to an age of three years and little change in strength between 3 and 5 years. A decided increase in compressive strength was also found when specimens which had been stored in air for 2 years and 7 years were transferred to moist storage conditions.

The specimens of Series A, D, P, S and W were made at the University of Illinois in the months of March and April, 1913. The test specimens were 6 x 6 in. cylinders. The concrete was 1 part Universal portland cement, 2½ parts sand, and 3½ parts broken limestone. The sand was screened through a ¼-in. sieve and was dry when used. The broken limestone was screened to pass a ¾-in.-mesh sieve and be retained on a ½-in.-mesh sieve. Several determinations showed that the stone, which was thoroughly damp when used, contained 4 per cent of water, and the water contained in the stone was taken into account in the calculation of the water used in the mixture. The concrete for all of the specimens was mixed by hand with a trowel, only enough concrete being mixed at one time to make one specimen. The specimens were molded in cylindrical metal molds. To distribute accidental differences the four specimens of each series tested on a given date were made on different days with the specimens of other series and ages.

The amount of water used to give concrete of the consistency designated as "normal" was 9.3 per cent by weight of the total materials (cement, sand and limestone); the concrete of normal consistency was drier than that ordinarily used in reinforced-concrete construction. In the mix of "dry" consistency 8.4 per cent of water by weight of total materials was used; for this mix water could be brought to the surface of the concrete with vigorous tamping. In the concrete of "wet" consistency 10.2 per cent of water was used; this consistency was not very wet as 13 per cent of water was required to give a mix which could not be used without considerable water separating from the concrete.

After 3 days the forms were removed and the specimens stored in damp sand or left exposed to the air of the laboratory until the date of test. The temperature of the storage room ranged from 60° to 75° F. during the months of March and April when the specimens were made and the temperature of the storage room throughout the year was usually about 65° to 75° F. The air of the storage room was more moist than would generally be found in an ordinary inclosed room. The temperature of the damp sand was generally about 6° lower than atmospheric temperature. In order to prevent absorption or loss of water so far as possible during the period of storage, the specimens of Series P were coated with paraffin before they were stored in the damp sand. The specimens were removed from sand storage 1 to 3 days before testing.

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* University of Illinois, Urbana, Ill.

The results of the tests are given in Table 1 and Figs. 1, 2 and 3.* It will be noted in Table 1 and Fig. 1 that the specimens of Series S (normal consistency, 9.3 per cent water), P (normal consistency, 9.3 per cent water, paraffined), D (dry consistency, 8.4 per cent water) and W (wet consistency, 10.2 per cent water), which were stored in damp sand, increased rapidly in strength up to an age of 1 year. The ratio of the strength at 1 year to the strength at 28 days averaged 1.94 for Series S, D and W; the ratio for Series P was 1.58. At ages greater than 1 year the rate of increase in strength for the specimens of Series W and P was relatively small, there being practically no increase in strength for these specimens from the 3-year to the 5-year age. At an age of 5 years the strength of the specimens of Series W and P was 2.6 and 1.9 times the strength at 28 days, respectively.

TABLE I.—COMPRESSIVE STRENGTH OF CONCRETE AT DIFFERENT AGES FOR VARIOUS CONSISTENCIES AND CONDITIONS OF STORAGE.

EACH VALUE, UNLESS OTHERWISE INDICATED, IS AN AVERAGE OF FOUR OR MORE 6 x 6-IN. CYLINDERS. PROPORTIONS, 1 : 2½ : 3½ BY WEIGHT.

Series.	Consistency.	Water Used in Mixing, per cent.	Storage Condition.	Compressive Strength in lb. per sq. in. at Age of									
				7 days	14 days	21 days	28 days	2 mo.	6 mo.	1 yr.	2 yr.	3 yr.	5 yr.
D	Dry	8.4	Damp sand	1751	2140	2658	2615	3332	3934	3945	4890	5340	4540
S	Normal	9.3	Damp sand	1390	1775	1816	1820	3063	3431	3765	4042	5116	5174
W	Wet	10.2	Damp sand	1103	1354	1623	1657	2410	3281	3757	3914	4278	4295
A	Normal	9.3	Air of lab.	1481	2061	2126	2116	2232	2049	2350	2189	2780 3455*	2774 4058*
P	Normal	9.3	Coated with paraffin, damp sand	....	....	....	2314	2521	3329	3671	4235	4340	4472

* Stored in damp sand when 2 years and 4 months old.

The specimens of Series S and D showed a considerable increase in strength from an age of 1 year up to an age of 3 years. The strength of the specimens of Series S like that of Series P and W remained nearly constant from the 3 to the 5-year age. The strength of the specimens of Series D, which, up to an age of 3 years, had been greater than that of the other series, fell off considerably at the 5-year age; no reason for this drop in strength is apparent and it is believed that the result is accidental. The strength of the specimens of Series S at an age of 5 years was 2.85 times the strength at 28 days.

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* The results of these tests for ages up to one year were reported in Vol. X, p. 435, of the Proceedings of the American Concrete Institute.

The specimens of Series A (normal consistency, 9.3 per cent water) stored in air attained nearly their maximum strength at the very early age of 14 days. From an age of 14 days up to an age of 2 years they gained but little in strength, the compressive strength over this period of time ranging from 2,051 to 2,350 lb. per sq. in. At an age of 3 years they showed some gain in strength and the strength at an age of 5 years was the same as that at 3 years. The strength at 5 years was 1.3 times the strength at 28 days.

When it was found that the strength of the specimens of Series A had not increased with age up to the 2-year age, one-half of the remaining specimens of this series was stored in damp sand when 2 years and 4 months old, to find the effect of damp storage at this advanced age. In Fig. 1 it is seen that at an age of 3 years the strength of the specimens so stored for the last eight months had increased considerably (675 lb. per sq. in.) over that of similar specimens stored in the air of the laboratory, and that at an age of 5 years the increase in strength of specimens so stored for the last 2 years and 8 months was 1,284 lb. per sq. in.—the strength of these specimens being 1.46 times the strength of similar specimens stored in the air of the laboratory. It is evident, therefore, that when the air-stored specimens were changed to storage under moist conditions further hydration of the cement

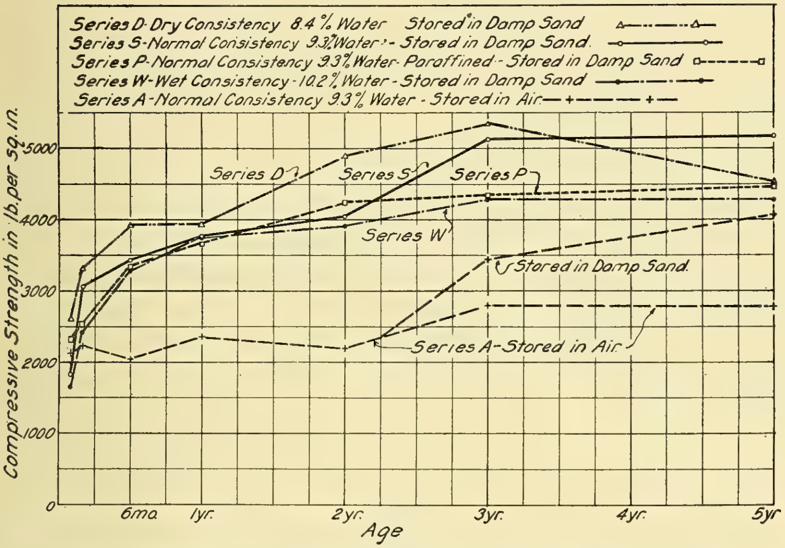

FIG. 1.—EFFECT OF AGE ON COMPRESSIVE STRENGTH OF CONCRETE OF DIFFERENT CONSISTENCIES AND STORAGE CONDITIONS.

occurred and the strength of the concrete increased markedly with the lapse of time and was approaching the strength of the specimens which had been stored in damp sand throughout the 5-year period.

In Fig. 2 are plotted the results of the tests of the specimens of Series S and A. As before stated, the specimens of these two series were of normal consistency, the specimens of Series S being stored in damp sand and those of Series A in the air of the laboratory. For ages greater than 28 days the specimens stored in damp sand were considerably stronger than the air-stored specimens. The ratio of the strength of the former to the latter ranged from 1.37 to 1.85 for ages from 2 months to 2 years, and from 1.85 to 1.87 for ages from 2 to 5 years. At the age of 5 years the ratio of the strength of the specimens of Series S to the strength of the specimens of Series A which were stored in damp sand when 2 years 4 months old was 1.28. It seems probable that with the lapse of time the strength of the latter specimens would approach more closely the strength of the specimens of Series S. A set of specimens stored in air and two sets stored in damp sand, having normal consistency but made under conditions somewhat different from the specimens of Series A and S, gave results at the various ages which tend to corroborate the results of Series A and S.

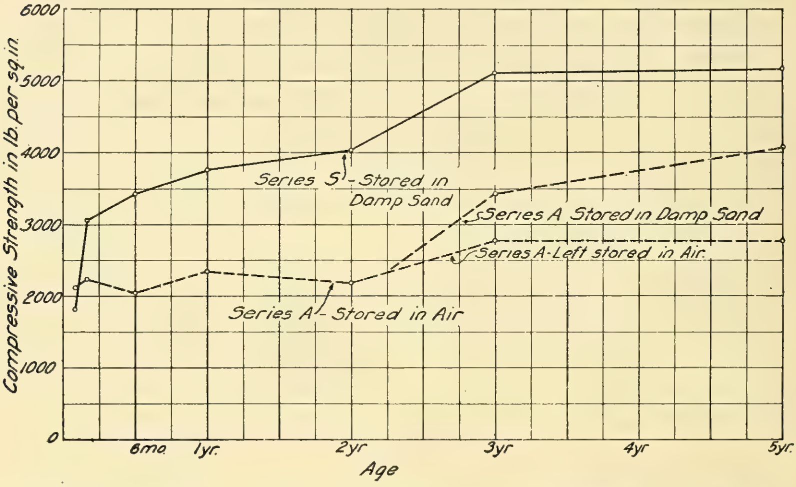

FIG. 2.—EFFECT OF STORAGE ON THE STRENGTH OF CONCRETE OF NORMAL CONSISTENCY AT VARIOUS AGES.

In Fig. 3 the average strength of the specimens of Series D, S and W stored in damp sand, and the strength of the air-stored specimens of Series A, are expressed in terms of the strength at an age of 28 days. For the sand-stored specimens the strength at 7 days is about 0.7 of the strength at 28 days, the strength at 1 year is about 2.0 times the strength at 28 days and the strength at 5 years is 2.5 times the strength at 28 days. For the air-stored specimens the strength at 7 days is 0.7 of the strength at 28 days, the strength at 1 year is 1.1 times the strength at 28 days, and the strength at 5 years is 1.3 times the strength at 28 days. It is seen that the condition of storage has a marked effect on the strength of the concrete at ages greater than 28 days.

It is probable that the strength of the stone aggregate itself affected the strength of the stronger specimens, for in these the stone aggregate was found to be fractured, while in the specimens of lower strength fractures of the stone were not common.

The specimens of Series Q were made under the direction of the late C. H. Cartlidge, Bridge Engineer of the Chicago, Burlington and Quincy Railroad Company, in March and April, 1910, and were stored in air in the company's laboratory at Aurora, Illinois. The tests at an age of 3 months were made at the company's laboratory; the tests at other ages have been made at the University of Illinois, shipment of each lot of 10 specimens being made two or three weeks before time of test.

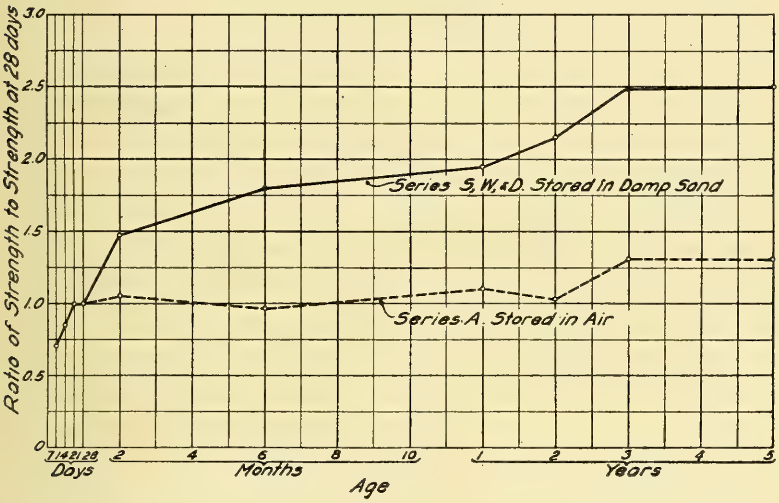

FIG. 3.—RELATIVE STRENGTH AT VARIOUS AGES OF CONCRETE STORED IN DAMP SAND AND IN AIR.

The test specimens of this series were 6 x 10 in. cylinders. The concrete was 1 : 4 mix, a bank run gravel being used as aggregate. The consistency of the concrete was reported to be similar to that ordinarily used in reinforced concrete construction. The results of this series of tests are plotted in Fig. 4. It will be seen that at the age of 6 months the concrete had attained nearly its maximum strength, only the test at 8 years giving a greater strength. In May, 1917, one-half of the remaining specimens of the series was stored in water and the other half was left stored as before, exposed to the air of the room. The tests made in 1918 when the concrete was 8 years old showed for the air-stored specimens a slight increase in strength over the strength at an age of 6 months. The specimens which were stored in water for 10 months showed an increase in strength of about 30 per cent over the air-stored specimens of the same age and an increase of 38 per cent over the specimens tested at an age of 6 months. It should be noted that for this series, as was the case with Series A, the growth in strength of the specimens stored under moist conditions is marked, even though the specimens were of considerable age at the time the change in storage was made.

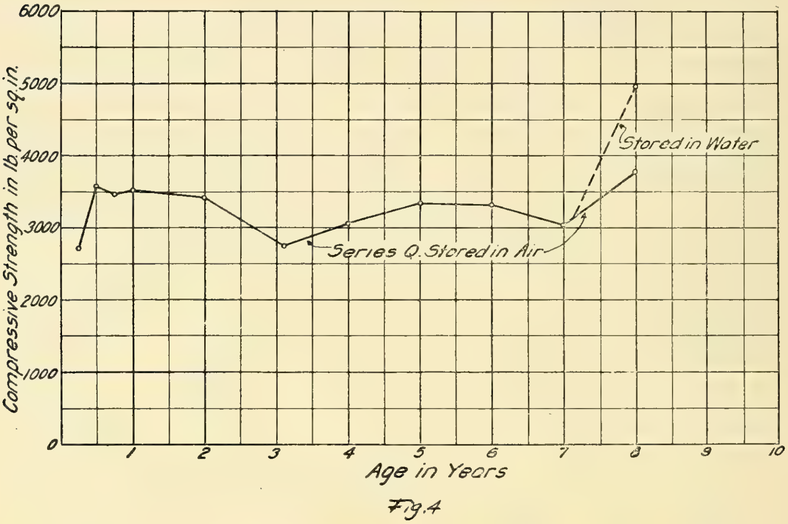

FIG. 4.—EFFECT OF AGE ON STRENGTH OF CONCRETE OF SERIES Q.

SUMMARY OF TESTS.

The tests of the specimens made at the University of Illinois may be summarized as follows:

1. The strength of the concrete which was stored in contact with moisture increased rapidly up to an age of 1 year; the increase in strength at ages greater than 1 year, although considerable, took place at a much less rapid rate.

2. The air-stored concrete attained nearly its final strength at a comparatively early age and gained little strength with the lapse of time.

3. The concrete which had been stored in air for a considerable time increased in strength greatly after it had been stored in contact with moisture so that further hydration of the cement could take place; the strength of the specimens stored in damp sand 2 years and 8 months after they were 2 years

and 4 months old was 1.46 times the strength of the specimens which remained stored in air for 5 years.

4. The strength of the concrete at an age of 7 days for both damp sand storage and air storage was about 70 per cent of the strength at 28 days; at an age of 1 year the strength of the concrete stored in damp sand was about twice as strong as at 28 days and the air-stored concrete was only 10 per cent stronger than at 28 days. At an age of 5 years the strength of the concrete stored in damp sand was about 2.5 times the strength at 28 days and the strength of the air-stored concrete about 1.3 times the strength at 28 days.

5. At ages of 3 and 5 years the strength of the concrete stored in damp sand was about 1.9 times the strength of the air-stored concrete.

The tests of the specimens made by the C., B. and Q. R. R. may be summarized as follows:

6. For the specimens stored in air the strength at an age of 8 years was slightly more than that at 6 months.

7. The strength of the specimens stored in water for 10 months after they were 7 years old was 1.3 times the strength of the specimens which remained stored in air for 8 years.

It seems apparent that concrete in structures exposed to air which is not damp will gain little strength beyond that attained at the earlier ages, in the portions where loss of moisture takes place, while concrete in contact with moisture or dampness will continue to gain in strength for some years.

DISCUSSION.

Mr. Bates. Mr. P. H. Bates.—In connection with the work of the Bureau of Standards at Pittsburgh on the properties of cement, we have been making concrete and storing it under various conditions. In general the results obtained in the great majority of cements, exhibiting different compositions, have been similar to those given in this paper. We have, in these series, used a number of unsound cements, very unsound; in fact, they have gone to pieces completely, and in some cases they have disintegrated to such an extent that we have had nothing on the glass plate. We have submerged some in water and at the end of five years they show no falling off of strength. We have made concrete from these same cements at the same time, but stored them in air, and they have disintegrated to such an extent that they crumble in the hand. Such a phenomenon is what I expect under the present theory of hydration of cement.

There is one other series of concrete which we have made of cement in the course of which the concrete was stored submerged in water, and some was submerged in a salt solution, and after a few weeks submerging in these two mediums, they were removed and dried out for a few weeks. This was continued for a long period and finally all the specimens were removed and stored in air. Under such conditions we had rather a curious result. The concrete stored in water alone, after being dried for about a year, showed an increase of almost 50 per cent in strength over what is shown in the wet condition. That, however, we do find sometimes. The small laboratory specimens stored in water and allowed to dry for a month or so will show a gain in strength. It is more a physical phenomena than a question of chemical hydration, or anything connected with the chemical hydration of cement.

TESTS OF STUCCO.

By J. C. Pearson.*

It is my purpose in this paper to present a brief review of the stucco tests undertaken some years ago by the Bureau of Standards and continued as a coöperative investigation on a larger scale during the last three years. So far as I am concerned personally, the investigation is a most interesting one, not only because in some respects it is a baffling problem, or a series of such problems, but also because it concerns many people in many walks of life. The improvement of stucco is therefore a worthy object, and it is our purpose to provide the architect with a material freed, if possible, from certain objectionable qualities which are now inherent only in our lack of knowledge of the material and the methods of its application.

The Original Tests, 1911.

The first stucco tests were started in 1911, and consisted of small panels of metal lath the majority of which were covered with cement and lime stucco in the approximate proportions of two-thirds part cement, three parts sand, and one-third part hydrated lime, by volume. A complete description of these panels was published in the "Proceedings" of this Institute for 1914, together with a progress report on the condition of the panels after two years' exposure. The test building on which these panels were erected is shown in Fig. 1. It is sufficient for our present purpose to recall that these panels were erected primarily for the purpose of determining the effectiveness of various treatments or protective coatings of the metal itself in preventing corrosion. These tests (which are still in progress) have demonstrated conclusively that galvanizing is the most effective treatment of metal lath for its preservation, and that a coating of asphalt or "dip" offers a sufficient degree of protection to the metal to ensure its satisfactory durability under average exposure conditions. Many of the painted or dipped lath panels are still in excellent condition after nearly seven years' exposure, notwithstanding the fact that the metal is exposed on the inside and the construction not entirely weatherproof.

Perhaps the most important lesson from these original tests, especially in view of later experience, was that corrosion of metal lath is one of the minor obstacles to the development of a successful stucco. Another interesting point in regard to these panels, which were given a sand float finish, is the pleasing color and texture which has developed through the action of the weather. The surface film of cement has in many cases been entirely removed, thus exposing the clean Potomac River sand of which they are composed. This is reassuring evidence that the appearance of a good stucco will improve with age, and therein lies nature's suggestion for the proper treatment of concrete surfaces.

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*U. S. Bureau of Standards, Washington, D. C.

THE TESTS OF 1915.

In these earlier tests, however, there was much that deserved, and to some extent received, a full measure of criticism. This was particularly true of some few stuccos which did not show up well in the tests, and of certain plasters which were used on interior panels erected at the same time.

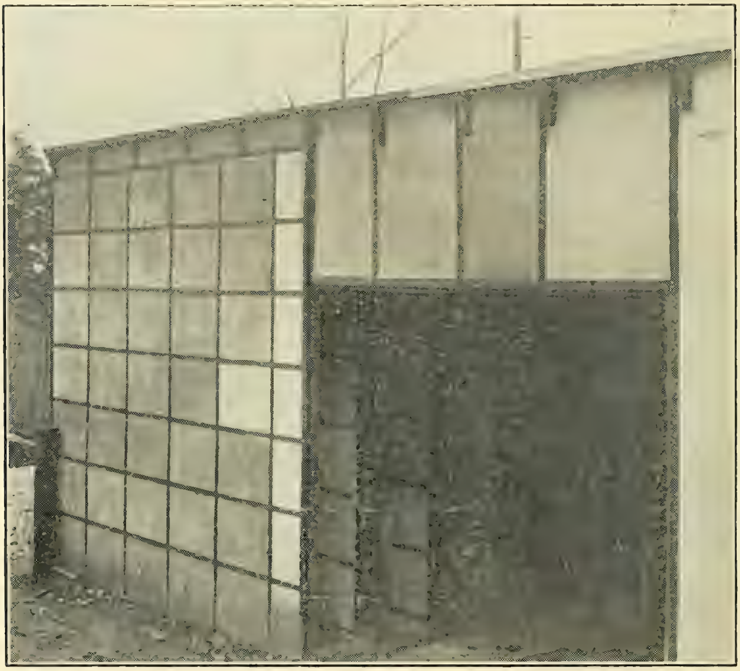

FIG. 1.—THE STUCCO TEST BUILDING ERECTED IN 1911.

The galvanized and painted metal laths in these panels, although exposed on the inside, show very little corrosion after 7 years.

Realizing that such criticism could only be avoided by the coöperation of commercial and technical associations, we organized an advisory committee of representatives of these associations, together with certain others who were by experience well qualified to serve on such a committee.

The members of this committee were unanimously of the opinion that stucco tests would have to be carried out on a large scale, under conditions that similate actual residence construction. The general opinion was that the test panels would have to be large enough to contain window or door openings, and to allow the plasterer the same freedom of action that he would have on a regular wall. The minimum size of panel that would meet these conditions was decided upon as approximately 10 x 15 ft. With this unit as a basis the committee drew up a comprehensive program for the test panels including practically all the common types of stucco construction. The specifications for each of these panels were carefully drawn in accordance with the recommendations of those most familiar with the type of construction in question, and were finally approved by the entire committee before becoming a part of the program. Without going too far into details, I desire chiefly to call your attention to the fact that nearly a year was spent on this program, and that it was prepared with all the assistance we could obtain from experts familiar with stucco specifications.

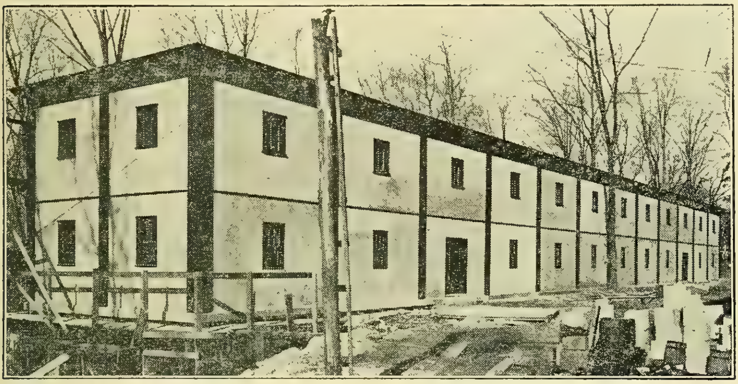

FIG. 2.—THE STUCCO TEST BUILDING ERECTED IN 1915.

Its walls are composed of 56 large test panels representing practically all the common types of residence construction.

The panels were erected during the summer and fall of 1915 and formed the exterior walls of the test structure shown in Fig. 2. The first progress report on these panels was prepared in 1916 and was eventually published as Technologic Paper No. 70 of the Bureau of Standards. This paper, as many of you are aware, describes in detail the construction of the test panels, and embodies a report on their condition in the spring of 1916 when the panels were about six months old. The impression created by this report has not been entirely favorable to stucco, especially when we take into consideration the extreme care with which the program was prepared and the constant supervision and inspection under which the work was carried out. This impression has been conveyed by the fact that in this report is recorded much of the detailed data gathered together primarily for the information of the committee in its study of the behavior and peculiarities of stucco. The committee has since expressed the opinion that much of this detail might better have been omitted, because it has tended not only to confuse the reader, but has to some extent proved misleading.

To illustrate this point, there are shown in Fig. 3 certain of the test panels which, from a moderate distance, have the normal appearance of satisfactory stuccos. Fig. 4 shows the same panels after spraying with water and allowing the surfaces partially to dry. This serves to expose in a startling manner the numerous fine and otherwise invisible cracks, which are usually present in greater or less degree in all stuccos. What I wish to emphasize particularly is that we should not be unduly alarmed over such demonstrations, for we have ample evidence that these small defects do not contribute appreciably to the failure of stucco. They may become an injury to the appearance, however, in the gradual accumulation of soot and dirt, which makes them more or less conspicuous, especially on a surface of fine texture.

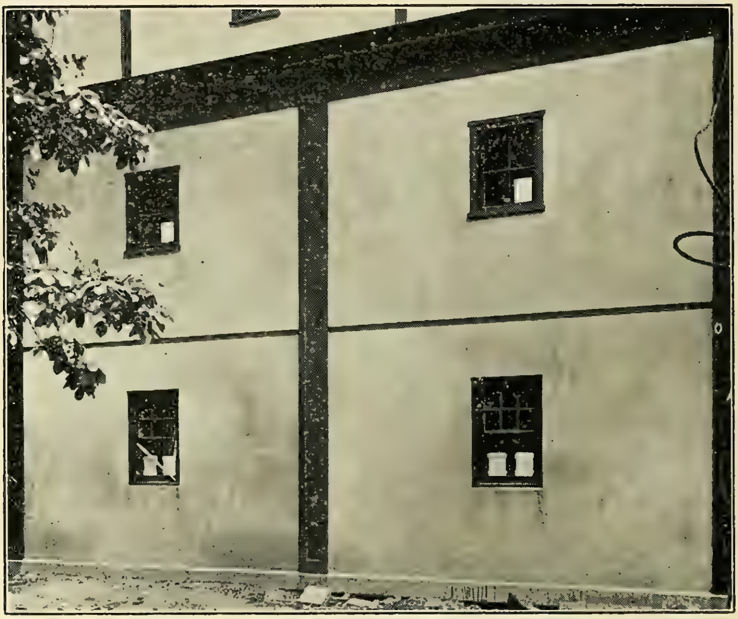

FIG. 3.—FOUR OF THE 1915 TEST PANELS FROM A DISTANCE OF ABOUT 25 FEET.

As stated in the preceding paragraph a full description of these panels is given in Bureau of Stanards Technologic Paper No. 70, and I will not attempt to give more than a brief description of the various groups into which the panels fall according to the bases on which they are applied.

Referring again to Fig. 2, the two ends of the building are of frame construction, the south end shown containing four panels of gypsum plaster board covered with a stucco composed of one part cement, three parts sand and one-tenth part by weight of hydrated lime. This was the accepted standard mixture more frequently used than any other on practically all of the various bases. These plaster board panels were removed in 1917, when it became necessary to convert the test structure into a laboratory, and at that time were approaching failure. Their appearance is shown in Fig. 5, which, however, does not show the full extent of the cracking, nor the very considerable warping and pulling away of the stucco from the base, especially near the horizontal cracks.

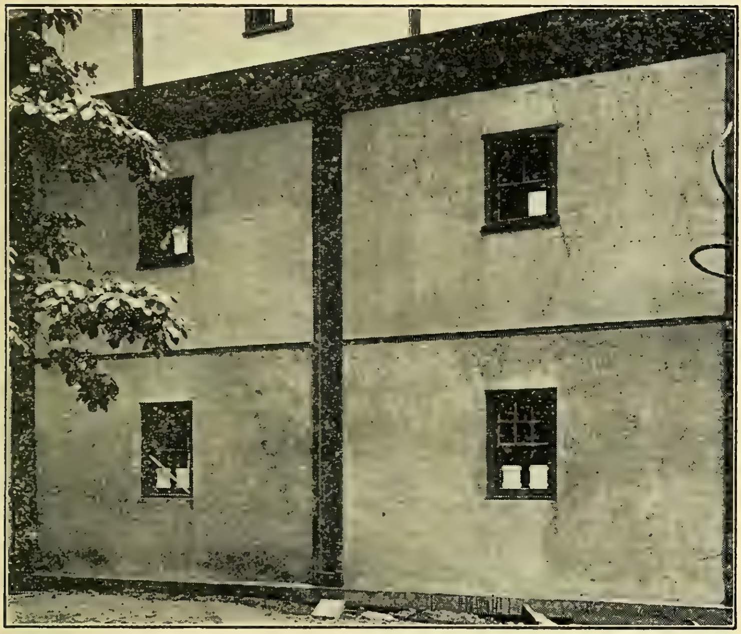

FIG. 4.—THE FOUR TEST PANELS SHOWN IN FIG. 3, AFTER WETTING.

Note the fine lacework pattern of the craze cracks in the lower panels. This is neither an alarming nor an unusual condition.

The corresponding four panels on the north end of the building are of metal lath, the back plastered type of construction, and one of them was plastered with this same mixture, one cement, three sand, one-tenth hydrated lime. This is the one panel on the entire building which, in 1917, was entirely free from cracks of any sort, and is therefore the only panel on the entire building to receive a 100 per cent structural rating. This panel is shown in Fig. 6.

The east and west walls of this test structure are of heterogeneous construction, for the most part masonry of some sort on the first story, and entirely of frame sheathed with 8-in. diagonal sheathing on the second story (with the exception of one panel). The first story contains six panels of monolithic concrete, surface treated in various ways before plastering, nine panels of terra cotta tile, one panel of concrete block, three panels of brick, and three panels of gypsum block. The two door panels shown in Fig. 2 are of frame construction covered with Bishopric Board nailed direct to studs. The second story contains fifteen metal lath panels covered with a variety of stuccos, three panels of wood lath plain lathed, five panels of wood lath counter lathed, and one panel of Clinton welded sheathing stapled direct to studs. Regardless of the distribution of stuccos, the concrete panels as a group have the highest rating, the plaster board panels the lowest. In these

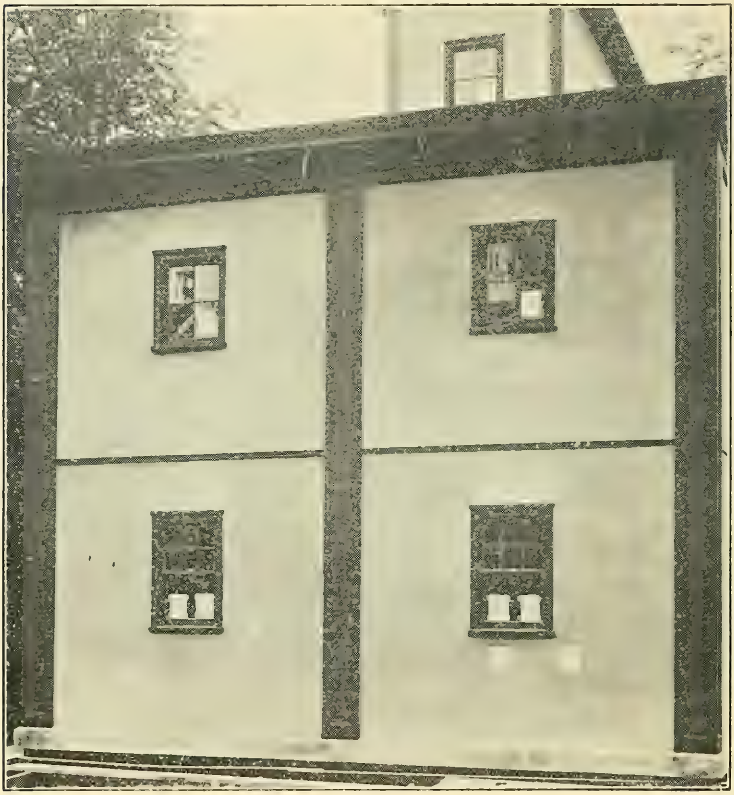

FIG. 5.—PLASTER BOARD PANELS ON SOUTH END OF TEST STRUCTURE.

These are approaching failure, caused by the use of a cement stucco on a weak base.

tests, the back plastered metal lath panels probably take second place, and the gypsum block panels are near the bottom of the list. Further than this one would hardly dare go toward a definite decision as to the comparative merits of different bases, and with even greater hesitancy one might attempt to select the most satisfactory stuccos. Those of us who are most familiar with the details of these panels were considerably nonplussed over the developments, the more so because there appeared to be so little in common to bases or stuccos of a given kind that would serve to indicate where improvement lay.

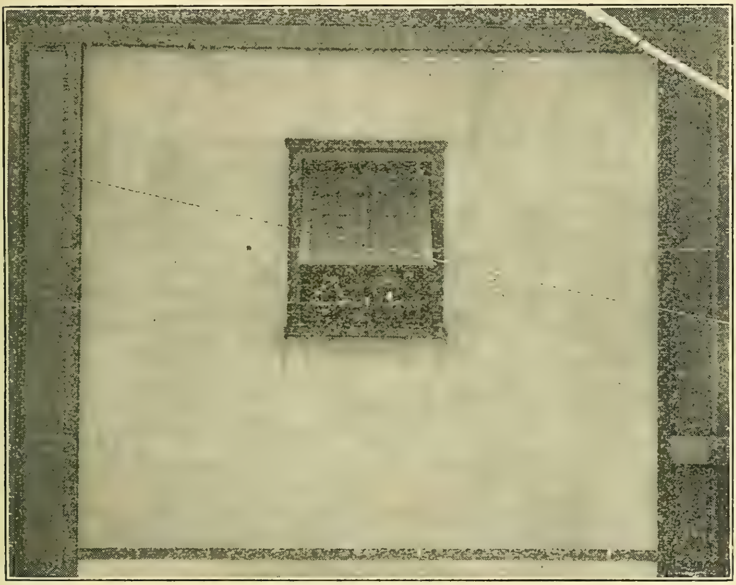

FIG. 6.—PANEL NO. 15 ON NORTH END OF TEST STRUCTURE.

This back-plastered cement stucco panel on metal lath is in perfect condition.

METHOD OF FINISHING CRITICAL.

There were certain outstanding facts, however, which furnished suggestions for future work and these I will enumerate briefly before describing the panels erected the following year, that is, in 1916. In the first place there was evidence that the prevalence of craze and map cracking on most of the 1915 panels was due to the method of finishing. It has been specified that a sand float finish should be used, and that this finish should be given in the shortest possible time after laying on the finish coat. The purpose of this was to avoid disturbing the so-called initial set of the cement. In consequence the very great majority of the panels were floated when too soft, which resulted in bringing to the surface a rich mixture of cement or cement and lime, subject to high shrinkage upon drying out. We believe now that this accounted largely for the general and early appearance of fine cracks on most of the panels, with some contribution also from the use of too rich mixtures.

Another peculiarity that impressed itself upon us was the appearance of large and prominent cracks on all the panels over diagonal sheathing. These cracks invariably first appeared off the corners of the windows and running across the direction of the sheathing, and there is no question that they were brought into prominence, if not actually produced, by the shrinkage of the sheathing. The fact that these characteristic cracks did not appear on the back plastered panels, and that the latter were largely free from prominent cracks, resulted in a higher rating for this group than for the metal lath panels over sheathing.

One other consistent development in these 1915 panels is of special importance, viz., It was noted that the stuccos on the plaster board and gypsum block panels and on the monolithic concrete panel coated with bituminous waterproofing were in poor condition, especially after they had passed through the second winter. All of these were stuccoed with the standard mixture of one part cement, one-tenth part hydrated lime and three parts sand, with only slight modifications in the method of applying. The results obtained on these panels indicate, not that this stucco is bad, but that the combination of this stucco and a weak base is bad. Field observations which I had an opportunity to make last year demonstrated this fact conclusively, that a strong cement stucco on a weak base is a common cause of stucco failure, and in practice this seems to occur most frequently in the application of brown and finish coats much higher in cement than the scratch coat, which logically should be the strongest portion of the stucco. The explanation of this failure must be sought in the well-known shrinkage of cement mortar upon drying out, and in the subsequent movements caused chiefly by varying moisture conditions.

THE TESTS OF 1916.

In laying out the program for the second year's work, it was found most feasible to erect a sort of monitor or penthouse, providing 22 additional panels, on the roof of the original building. This of course precluded all but frame construction. The back plastered panels had shown up so well in 1915 that it was decided to include a larger proportion of these in the new layout, and only one-half the monitor or annex, as it is more commonly called, was sheathed. To minimize the shrinkage effects 6-in. diagonal sheathing was used, alternating in direction on adjacent panels. It was decided also to change the method of finishing, partly in the use of less water on the under coats, partly in waiting for the stiffening of the finish coat to develop before finally floating. This procedure, together with the use of a number of leaner mixtures, was adopted in the hope that craze and map cracks would be largely overcome. A number of special features were also included in the construction of certain panels with a view to minimizing the effects of movement of the wood frame. The annex was completed in October, 1916, and is shown in Fig. 7.

I will first outline the general results obtained, and then describe a few of the more interesting panels in this collection. We had hoped that the panels over sheathing would at least show an improvement over the similar panels erected in 1915. So far as the sheathing was concerned, however, they did not; the same peculiar cracks developed as before, and there was a period soon after the panels were completed when one could determine the direction of the underlying sheathing from the appearance of the cracks. These cracks invariably first appeared perpendicular to the direction of the sheathing; all the panels suffered in this respect, and if anything they are in worse condition as a whole than those of the year before. In this group the stuccos on wood lath are especially bad, and those containing a high percentage of lime are in decidedly worse condition than those containing a highpercentage of cement. So far as this investigation has gone, then, we have about decided to condemn diagonal sheathing, unless there is some assurance of getting seasoned wood for the purpose, or of finding some other means of counteracting the shrinkage effects.

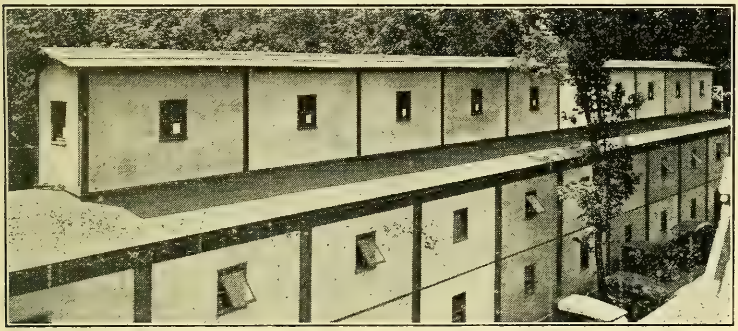

FIG. 7.—ANNEX ON ROOF OF ORIGINAL TEST STRUCTURE, ERECTED IN 1916.

A very great improvement in finish was obtained on these panels and as a result craze cracks were largely, but not wholly eliminated. The larger patterned map cracks were also very greatly reduced, and this was particularly noticeable in the case of those panels which were stuccoed with radically lean mixtures. One of these is a back plastered metal lath panel stuccoed with a mixture of one part cement and six parts limestone screenings. This panel is shown in Fig. 8. It is in perfect condition and waterproof even when subjected to heavy driving rains.

There are several panels on the annex in very satisfactory conditon and as some of these are of unusual construction I will describe them briefly. Three of the panels were especially constructed in such manner as to free the stucco as much as possible from the movements of the frame. One of these was over wooden studs to which was attached Clin-Trus furring. This is a sort of welded wire frame to which the lath is tied, and which offsets the stucco some 2 in. from the face of the studs. The purpose in using this material was to take advantage of a certain flexibility of support for the stucco, so that if the studs twisted or buckled, the strain would not be transmitted directly to the stucco. A few inconspicuous cracks have appeared on this panel but on the whole it is in very good condition. On another of these panels $\frac{3}{8}$-in. Hy-rib lath was wired around the studs, the lath being entirely supported in this way without the use of nails or staples. This panel is without cracks and in perfect condition. The third panel was an adaptation of the Flagg wire partition to an exterior wall. Stout supporting wires were fastened at the top and bottom of the panel only, and then drawn taut by tying them laterally at intervals in such manner as to form a large diamond mesh pattern, upon which metal lath was wired to support the stucco. This panel contains one diagonal crack only running from opposite corners of the window to the frieze and watertable. This crack appeared quite suddenly about six months after the panel was completed, and suggests that possibly the supporting wires, which are necessarily under very high tension, might have slipped in one or more places. In these three panels also a special type of expansion flashing was used around the trim of the windows, designed

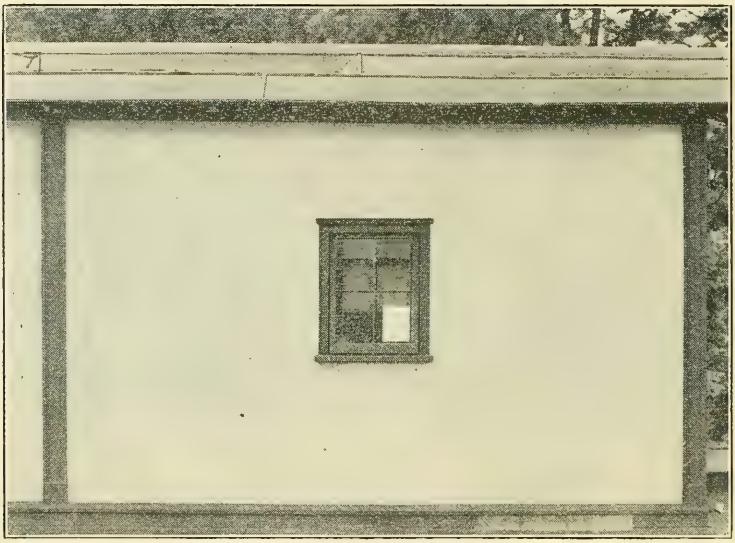

FIG. 8.—PANEL NO. 66, ERECTED IN 1916.

This panel is of back-plastered metal lath, stuccoed with a mixture of 1 part cement and 6 parts limestone screenings. It is waterproof and in perfect condition.

to prevent the formation of radial cracks at the corners by the pressure of the wooden trim when expanded by moisture. So far as one can judge from the condition of these panels, the flashing has been effective, and is well worthy of further investigation.

Two panels of Bishopric Board were included among the 1916 panels, one covered with cement stucco, one with a magnesite stucco. The former showed the same characteristics as the original panels of this type in the development of a number of large cracks, the latter was in perfect condition during the first year of its existence but now shows some fine cracks near the bottom. The magnesite panel is shown in Fig. 9.

Two additional gypsum plaster board panels covered with cement stucco were also included in these tests, and, as anticipated, showed essentially the same serious deterioration as the original panels of this type. The results obtained from these panels eliminate plaster board from further consideration as a base for cement stucco. The combination is wrong, both in theory and practice.

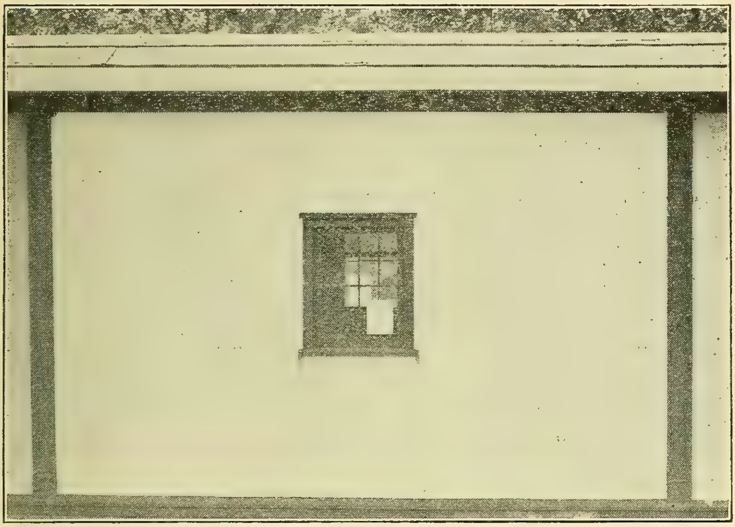

FIG. 9.—PANEL NO. 62. MAGNESITE STUCCO ON BISHOPRIC BOARD.

The panel is in very satisfactory condition but shows slight deterioration at the bottom.

One panel was erected on pressed steel studs, covered with $\frac{3}{8}$-in. Hy-rib lath and stuccoed with a mixture of one part cement, four parts sand and one-tenth part hydrated lime, by weight. We expected much of this panel and it is really in very good condition, but a number of fine cracks can be detected on close observation.

The only other panel worthy of special mention is one of ribbed lath, back plastered, and stuccoed with a mixture of one part blended cement and three parts sand. The blended cement was prepared in the laboratory by grinding together equal parts of Potomac River sand and normal portland cement to about 90 per cent through the No. 200 sieve. The mixture, therefore, contains one part portland cement and seven parts sand, of which one part is very finely divided. This panel is one of the finest in evenness of color and texture, but shows a single fine crack over the bridging. This I believe was due to carelessness and could easily have been avoided. Certainly the possibilities of blended cement should not be overlooked in future experiments, for the material possesses ample strength, and provides the plasticity which is usually wanting in lean mixtures.

SUMMARY OF 1916 TESTS.

The deductions from the 1916 tests may be summarized as follows:

1. Diagonal sheathing of unseasoned wood is apparently an unsatisfactory backing for stucco. With only unseasoned wood available horizontal sheathing would appear to be better construction, provided sufficient bracing of the wood frame is assured.

2. Back plastered construction appears to be best for frame structures, so far as the integrity of the stucco is concerned. This carries with it, however, a need for fuller information regarding the insulating qualities of walls so constructed.

3. Lean mixtures promise better cement stuccos, provided the necessary plasticity and density can be maintained by proper grading of the aggregate.

4. There is still need of further information as to the value of wood lath and high lime stuccos. To date the tests indicate that wood lath is not as satisfactory as metal lath, and there is no conclusive evidence that a modern hydrated lime stucco will endure satisfactorily in severe climates.

THE TESTS OF 1917–18.

Last year the committee was unanimously of the opinion that many of the original panels had served their purpose and it was planned to replace some of these with new panels embodying the suggestions obtained from the first two years' tests. The outbreak of the war, however, prevented the carrying out of the program, and progress has consisted mainly in the extension of our experimental work to include certain temporary structures erected at the Bureau of Standards as emergency testing laboratories. Three such buildings of frame construction have been erected, and others are contemplated. One of these temporary buildings is approximately 25 x 50 ft. x 14 ft. high covered with Bishopric Board nailed direct to studs and stuccoed with a cement stucco of one part cement, two and one-half parts sand and ten per cent hydrated lime by volume. Special attention was given to the bracing and framing of this structure, which was done under the supervision of the manufacturers of Bishopric Board. The second of these buildings is a structure similar to and alongside the first, covered with ⅜-in. Hy-rib metal lath,—back plastered and stuccoed with a mixture of one part cement and three parts sand. These buildings offer a good opportunity to compare two radically different types of construction, and while the second is decidedly better than the first, neither of them is as good as we had hoped. From a critical and investigative point of view, neither of them can be considered wholly satisfactory, and we are bound to admit that further study of the methods of applying stucco to frame construction is necessary before an absolutely reliable specification can be written. These two buildings are shown in Fig. 10; the one on the left is of Bishopric Board, the one on the right is of metal lath.

The third building is approximately 30 x 70 ft. x 27 ft. high, and nearly a repetition of the construction of the second, viz., $\frac{3}{8}$-in. Hy-rib lath, ribs inward, on 2 x 6-in. studs, bridged and corner braced. Special pains were taken to intermesh the ribs of adjacent sheets of lath, large-headed nails were driven at every crossing of ribs and studs, and a wire tie was added at every interlocking rib between studs. The result was as nearly a homogeneous fabric as could well be obtained on which to apply a stucco. The stucco was composed of one part cement and three parts sand, machine mixed, back plastered, and given a lightly scraped finish after floating. This, as an experiment in texture, was not wholly satisfactory on account of the lack of uniformity obtained. The sand grains were to have been cleanly exposed by acid washing, but fine cracks were detected before this treatment was applied, and the acid washing was abandoned. At the present time this building is quite satisfactory in appearance as shown in Fig. 11, but numerous horizontal cracks may be detected on close examination, and a few of the characteristic cracks off the corners of windows and doors have developed. In one sense

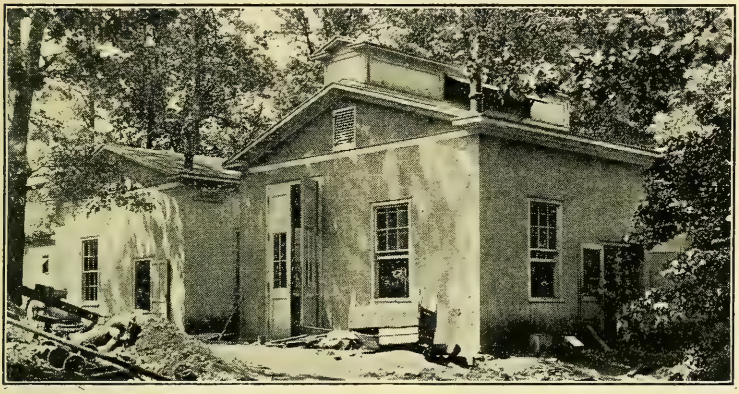

FIG. 10.—TWO EXPERIMENTAL STUCCO BUILDINGS.

One on the left is of Bishopric Board, the one on the right is of metal lath back-plastered. The latter is in better condition than the former, but neither is free from objectionable defects.

this experiment was quite successful as it seems to demonstrate sufficient reinforcing value of the ribs of the lath across even six or eight inches of vertical grain of underlying wood. The weakness appears to be rather in the vertical direction, in which the reinforcing value of the lath is almost negligible.

In the very near future, we shall have an opportunity to apply experimental stuccos on three small penthouses of tile on the roof of the new laboratory shown in the background of Fig. 10. As a tile base is particularly adapted for the use of lean mixtures, it is planned to cover two of these with such mixtures, and to carry out a radical experiment in texture on the third.

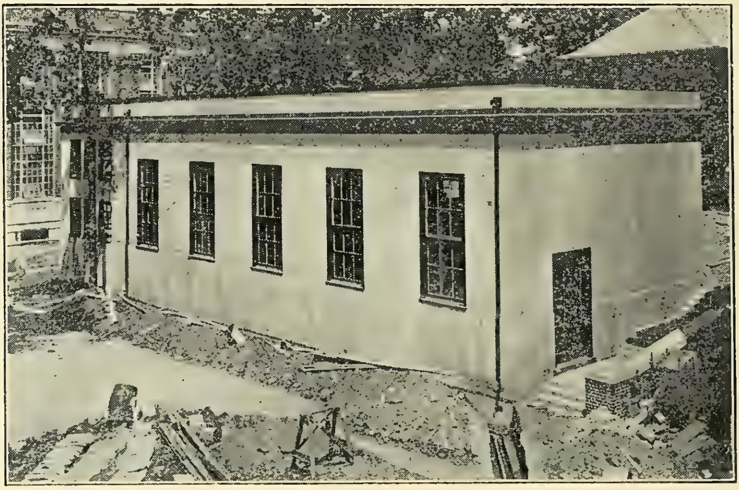

FIG. 11.—EXPERIMENTAL STUCCO BUILDING OF BACK-PLASTERED METAL LATH.

Is in good but not perfect condition. Fine cracks have developed which indicate a weakness peculiar to the type of lath used.

I have attempted to outline in brief space the experimental work accomplished thus far in our stucco investigation. The results obtained to date indicate that we have only begun the work, if we intend to carry it through to a successful finish. This we do intend, and while progress may be delayed during the period of the war, we are taking advantage of every opportunity to get information from the field, and when the tests are again resumed along the lines originally planned, we hope to have many suggestions on which to base the new work.

Magnesite stuccos are now coming into prominence and we are watching these with great interest, for the main question in regard to them is one of permanence. These stuccos have properties which distinguish them from portland cement, or cement and lime, at early periods, but the test of time is wanting to demonstrate their superiority over the more commonly used materials. Personally I believe that portland cement stuccos have great possibilities, for their permanency has already been satisfactorily demonstrated. The principal defects in these stuccos are due to their peculiar volume changes, and a careful study is being made of these movements for a better understanding of them. It is hoped that by proper combination of the ingredients, and by proper methods of mixing and application, these defects will be gradually overcome.

The ordinary constituents of cement stuccos are such that by careful and appreciative manipulation they themselves may be used to produce highly pleasing effects of color and texture. Progress along these lines is toward the ultimate in stuccos of this type, and I feel that as our knowledge of the nature of them improves, it will carry with it the elimination of such defects as we have not yet been able to overcome.

DISCUSSION.

A Member. A MEMBER.—What do you mean by magnesite stucco?

Mr. Pearson. MR. J. C. PEARSON.—I refer to the proprietary materials of which the stucco is composed. The basis is magnesium oxide and chloride.

Mr. Lesley. MR. R. W. LESLEY.—Do I understand that that particular panel is beginning to crack a little now?

Mr. Pearson. MR. J. C. PEARSON.—Yes, there are several cracks.

Mr. Lesley. MR. R. W. LESLEY.—The reason I ask is that there is a remarkable bridge in France built with portland cement quite high in magnesia, and from that bridge many of our specifications date, so far as the magnesia content is concerned. The same trouble occurred in that bridge that you recite here. The masonry was at first perfectly sound, but at the end of two or three years it began to disintegrate. The trouble with all these magnesia compounds—I dealt with several of them years ago in building artificial stone houses and things of that kind—is that they last perfectly for a year or two but ultimately they begin to go. It is a very dangerous compound, because it is the over burning of the magnesia, which disintegrates after a much longer period than the lime compound in the cement.

Mr. Pearson. MR. J. C. PEARSON.—I will say this in justice to the manufacturers of this product; I have seen houses in very good condition indeed, after five years exposure, I do not know what we may expect but I do believe that the design will count largely in this type of stucco, that is the protective features will count largely. Possibly if we give it as gentle an exposure as is possible under the architectural requirements, this stucco will be satisfactory. I might say, too, that we have not a right to condemn them on laboratory tests, and we do not. The only thing that makes us hesitate is that we know that they will not stand the laboratory exposure, such as we give in ordinary tests of portland cement and it leads us to hesitate about predicting how long these stuccos will last.

A Member. A MEMBER.—How did this cement stand the storm conditions?

Mr. Pearson. MR. J. C. PEARSON.—Disintegration was not rapid. A single severe exposure or a few exposures seemed to have no effect.

Mr. Boyer. MR. E. D. BOYER.—You did not say, as I recall, what that was put on. I would like to ask if that was put on metal lath; is it not a fact that it was not fireproof?

Mr. Pearson. MR. J. C. PEARSON.—I think you can put it on anything. Personally, if I were to choose, I would prefer a wood lath as best for it on account of its tendency to corrode metal. I think it is fireproof.

Mr. Hatt. MR. W. K. HATT.—We had an experience in the laboratory a few years ago. A firm in Chicago was making this product. They got it out of a lake and made quite a few tests on it and it had remarkable physical properties for strength and elasticity. We also had a fireproof test on it and the result was we exploded our furnace and it cost us more than we got out of the test.

MR. J. C. PEARSON.—The chief indications from the 1916 panels on the pent house were that there was great promise in the lean mixtures if we can find some way to apply them practically with a good mix and a good workman. I believe we have something that is going to be a decided improvement over the normal stucco applied according to the present specifications. On those panels which were over the diagonal sheathing, we had some comparative tests between high lime stuccos and high cement stuccos, and perhaps a better comparison between wood lath and metal lath than on the original building, and while we had no conclusive results, the condition of the panels on the pent house at the present time indicates that the high lime stucco on wood lath are in probably the worst condition of any on that frame structure, and the wood lath panels are decidedly inferior in condition generally, on the average, we will say, to those of the metal lath. That again might need some modification, because I believe the best we can get from wood lath would be to require better nailing of the laths, and, if possible, a narrower lath than is usually used for inside plastering and wider keys. I do not know whether that anchorage provided is sufficient for another portland cement stucco or a very strong portland cement stucco. Our panels indicate that it is not, but we did not do the best we could in the application of the wood lath. I think, really, because wood lath is so widely used, that we should try to get what we can out of that, although, personally, I have not the confidence in it that I have in the metal lath. **Mr. Pearson.**

A MEMBER.—Did you notice that the cracks on Bishopric lath followed the joints of the lath or across the face of the sheets? **A Member.**

MR. J. C. PEARSON.—In our panels we could not tie up the position of the cracks with the joints in the laths with any regularity enough to make any statement. The cracks generally tend to come parallel to the lath, perhaps more noticeably than any other way, but also more or less zigzag and sometimes perpendicular without being over the joints. **Mr. Pearson.**

A MEMBER.—The Bishopric lath, I understand, come in about 4-ft. sheets. Did you make joints? **A Member.**

MR. J. C. PEARSON.—We did on the test panels. **Mr. Pearson.**

MR. N. G. HOUGH.—I would like to ask with reference to your conclusion that there is no conclusive evidence that a modern hydrated lime stucco will endure satisfactorily in severe climates. Do you refer to wood lath or other background? **Mr. Hough.**

MR. J. C. PEARSON.—I did not have the background in mind; I would say any background on which you can get the best result. **Mr. Pearson.**

MR. N. G. HOUGH.—If that is a general statement, I do not think it is justified, because is it not a fact that in your field of investigation you came across lime stucco jobs that were entirely satisfactory, that is, satisfactory from our viewpoint at the present time? **Mr. Hough.**

MR. J. C. PEARSON.—I remember just one that I would call satisfactory. There were very few of them in existence, as far as I know. **Mr. Pearson.**

MR. N. G. HOUGH.—That is quite true in latter years, but the one you have in mind, the one in Pittsburgh I believe, was not that quite satisfactory? **Mr. Hough.**

MR. J. C. PEARSON.—Yes sir. **Mr. Pearson**

Mr. Hough. MR. N. G. HOUGH.—The reason for that, in my opinion, is that some of the necessary fundamental qualifications, such as the providing of water table drifts, etc., were properly carried out on that job.

Mr. Perrot. MR. E. G. PERROT.—I would like to ask a question in regard to the speed of putting up stucco, especially in cold weather. Will the use of calcium chloride be beneficial or detrimental in the long run to stucco work, especially with the metal lath?

Mr. Pearson. MR. J. C. PEARSON.—I think it would be detrimental to the metal lath. About a year ago we exposed some slabs on the roof. They were cast flat in such form that the density would be greater than the density of a stucco, and in those metal lath samples which were imbedded. We found quite appreciable corrosion at the end of the year, so that it looked rather uncertain. I think the best thing to do would be not to apply portland cement stucco in cold weather.

SOME PROBLEMS IN DEVISING A NEW FINISH FOR CONCRETE.

By John J. Earley.*

Reinforced concrete has long been justified structurally but its appearance leaves much to be desired. In realization of its shortcomings in this respect, and in consideration of the importance of a work that would give to concrete an appearance commensurate with its structural value, an effort was made to acquire an intimate knowledge of what had been done towards this end. The search was discouraging and on the whole without profit. No technique, applicable to the jobs under consideration, had been developed in the surface treatment of concrete. The evidence showed that appearance had been an afterthought, a secondary matter, a desirable but rather impossible thing which should receive all the consideration possible after the structural problems had been solved.

An analysis of the problem of appearance indicates three principal factors; namely, form, color, and texture, all of which must be developed before reinforced concrete can be accepted, by architects, as a fitting medium for the expression of academic design. The degree in which reinforced concrete meets these three requirements is the measure of its liberation from its present restricted use. I hope that the time is not far distant when this view will be justified, when trained men will be meeting every requirement in form, color, and texture that architects may impose: in other words, I hope to see the day when a rendered drawing will supplement the specifications for every reinforced-concrete structure, and the structure will be a faithful expression of design.

About two years ago the United States through the Office of Public Buildings and Grounds, built a retaining wall on the west side of Meridian Hill Park, Washington, D. C. This park is to be a beautiful and formal garden on the hill which marks the meridian of Washington, D. C. The original plans called for a reinforced-concrete wall with a stucco finish, and certain decorative features which were to be cast in cement mortar and set in place. Extensive studies were made, including samples and full-size sections. Finally a unit of the wall was built in place, in concrete, complete in every detail excepting reinforcement. The result was a plastered wall, nothing more. The construction might just as well have been of brick, terra cotta, or metal lath on channel studs so far as the appearance revealed. The wall was without scale. It did not give the appearance of strength and size equal to its task as a retaining wall. The color was unsatisfactory. It was the cold gray cement color that has always been so objectionable. Every particle of sand was coated with cement and had no color value of its own.

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* Architectural Sculptor, Washington, D. C.

An effort was made to relieve the monotony of appearance by finishing various details in different texture. The piers were built of rusticated blocks the surface of which was grooved vertically with straight, symmetrical, V-shaped grooves about $\frac{3}{8}$ in. on centers and $\frac{1}{4}$ in. deep. The coping and border of the wall panels were sand floated. The panels themselves were pebble dash in which the pebbles were as nearly as possible one size, having passed a $\frac{1}{4}$-in. screen and having been retained on a $\frac{1}{8}$-in. screen.

At this point it was decided to set aside the methods of finishing employed in the studies and full-size model and to use others that would be more characteristic of the wall construction. It was then suggested that the forms be removed while the concrete was yet green and the surfaces brushed with steel brushes until the aggregate was exposed. Permit me to emphasize in an especial manner, that it was this decision to employ finishes characteristic of the nature of reinforced concrete that gave the interest to succeeding studies and resulted in the development of surface treatments as applied at Meridian Hill Park.

This method of treating the surfaces at once supplied the sense of strength and size that was lacking before. The wall was no longer a plastered one, but was reinforced concrete and nothing else, and it seemed big and strong enough to suit all the demands that would be made upon it.

A change took place in the color. The surface, which had been wholly of a cement gray, was broken in frequent spots by clean pebbles in their natural color which varied from white, to yellow, to light brown. The pebbles in a pile would be yellow. These spots relieved the gray of the cement to such an extent that they imparted to the whole structure a cream color which was a great improvement and a decided step forward.

The exposed pebble texture was used on the wall panels, the face of coping, and the rusticated blocks of the piers. The borders around the wall panels, and rusticated grooves on the piers were finished differently. The forms were tooth-chiselled and the latter were left untouched. The reason this was done was to accent the drawing. It was feared that if the same texture that was used on the panels were used everywhere, the structure would have too rough an appearance and the drawing would be lost.

The forms for the piers were plaster, shellacked on the inside to make them waterproof, and so made that they would fit inside rough wooden forms which were depended upon to support the concrete. The forms for the panels were constructed of dressed lumber and followed general practice. Separate strips, channelled out on the back, were used to form the drip under the coping and the groove around the large panels. These strips were so arranged that they remained in place when the forms were removed and the shrinkage of the wood in drying was depended upon to release them.

A pier and a panel composed one unit and every other one was poured, then the ends were coated with five ply felt and tar to form an expansion joint. This method was followed in order to give the first panels time to harden so that there would be less danger of breaking them when working on the adjoining panels.

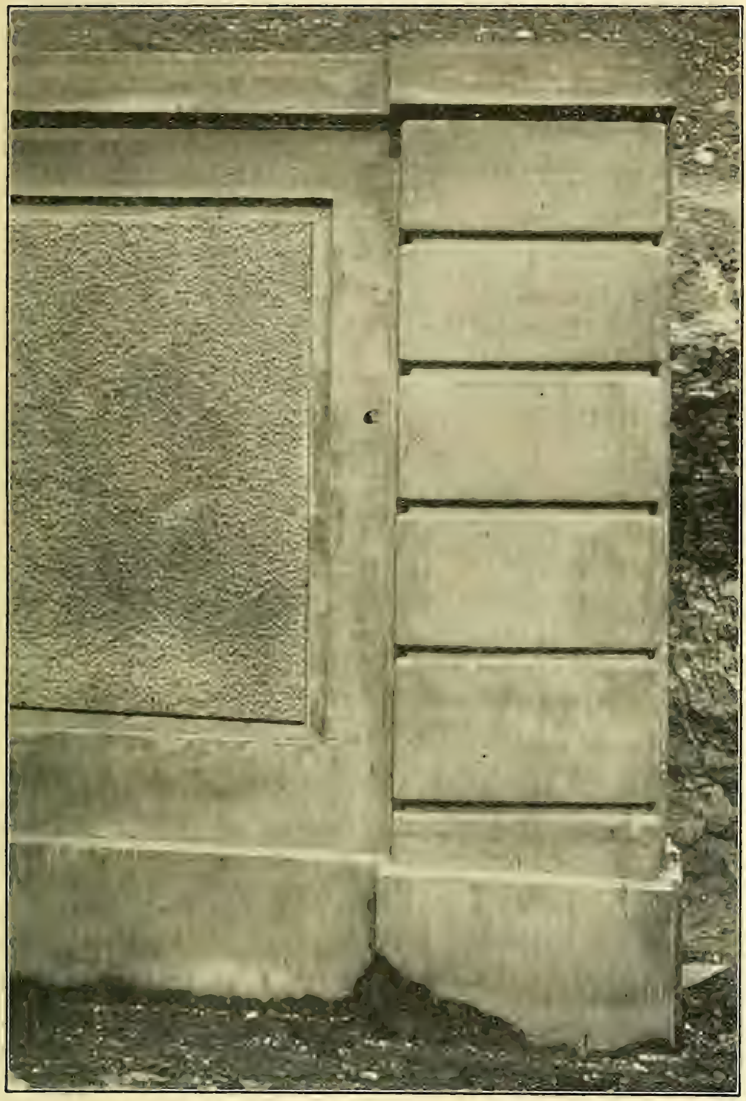

FIG. 1.—REINFORCED-CONCRETE WALL WITH STUCCO FINISH, AND PRECAST RUSTICATED PIER WITH VERTICAL GROOVES FOR TEXTURE.

It has the cold grey cement color of ordinary concrete. The textures are conventional and not characteristic of concrete.

The concrete mix was 1:2:4. The cement was a standard brand passing the tests of the Bureau of Standards, the sand was fairly clean river sand of the size known as concrete or torpedo sand, namely, everything that passed the ¼-in. screen on the dredge. The pebbles were of good quality, hard and reasonably uniform; the largest passed a 1¼-in. mesh and were retained on a ¼-in. mesh; however, the great bulk were, in size, such as would pass a ¾-in. mesh. This concrete was poured into the molds and its consistency was soft, so soft that it would flow readily.

After 24 or 48 hours, according to the condition of the cast, the forms were removed and the surfaces scrubbed with a steel brush until the aggregate was exposed as evenly as possible. The entire unit was then thoroughly washed with water from a hose. The seasoning was carefully watched and frequent wettings were used to prevent too rapid drying.

The cast cement trimmings were what is known as dry pressed castings, that is, one part of cement and 1½ parts of coarse sand are moistened so that the material will hold its shape when pressed together but not so wet that water can be squeezed from the mortar. This mortar is then tamped into plaster piece molds which can be removed immediately. These castings when well made are very sharp and clean, conforming to every requirement of form but their color and texture is limited to the color of the cement and the texture of the sand. From the foregoing an idea may be gained of the problem presented by the first retaining wall at Meridian Hill Park and how the conditions were worked out. The accompanying photographs will illustrate the character of its work, showing clearly how the conditions of form and texture were met, and while the color is not recorded this requirement was complied with about as well as were those of form and texture.

The second operation at Meridian Hill Park consisted of a balustrade 300 ft. long located on the upper level of the park, and an arched entrance with a flight of steps connecting the upper level and the street.

The technical progress made in the execution of this work and the great improvement shown in the results were largely due to a careful study and severe criticism of the first operation. So radical were the changes in methods and materials that I feel that I must pause here to make sure the criticism I am about to make of the first operation will in no way belittle it in your mind; for the first operation stands in no need of apology. It is a thoroughly satisfactory concrete retaining wall, rather better than was expected at the start, and while the second operation shows marked improvement, it is only a reasonable progress and must not detract from the merit of the first.

At the opening of this paper I spoke of the desirability of reinforced concrete being made to meet any requirement of form, color, and texture that the architect may impose. I had been encouraging the architects in this belief and the second Meridian Hill operation certainly reacted vigorously. The 300-ft. balustrade was detailed without restriction. The entrance was a classical arch flanked by pilasters and surmounted by a complete entablature. In designing no concession was made to the material, but every requirement of form was insisted upon. The photographs show the sharpness of line and the refinement of contour that characterize all the details. The same method of surfacing was employed as on the first wall; namely, that of removing the forms while the concrete was green and scrubbing the surfaces with a steel brush. A change of texture was made to accent the drawing but in this case the effect was obtainable by changing the grain of the texture rather than by changing the nature of the texture.

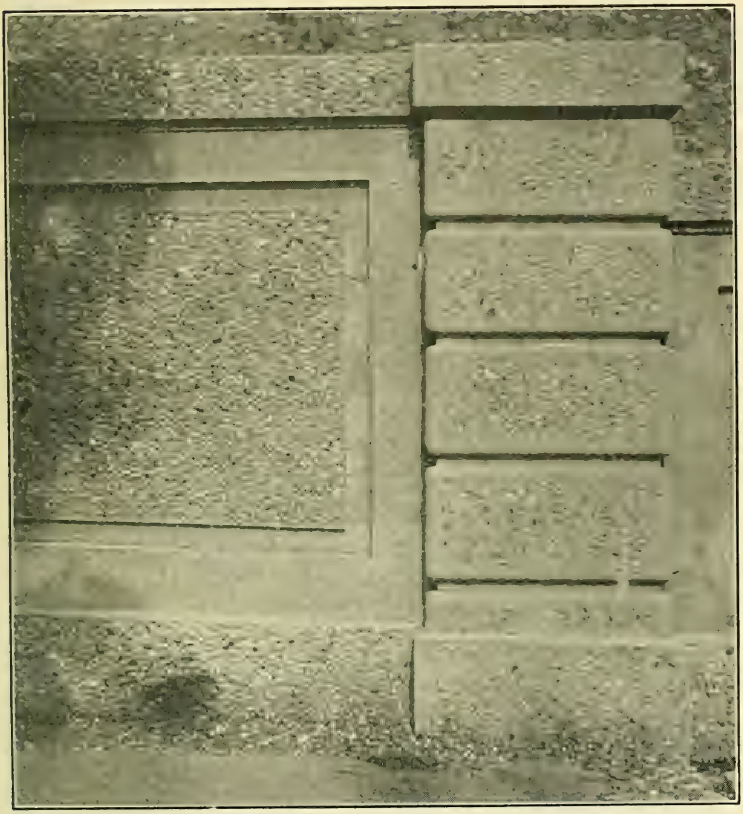

FIG. 2.—REINFORCED-CONCRETE WALL AND PIER SIMILAR IN DETAIL TO THAT SHOWN IN FIG. 1.

Texture and color obtained by exposing aggregate. Note that the surface treatment is characteristic of concrete, as opposed to that in Fig. 1.

The forms for the balustrade and arch were an art in themselves and the difficulties encountered were most discouraging. Morning after morning the previous days work was broken up and thrown away. Sometimes the average was one cast out of five, sometimes a little better. In the beginning it was only a great faith in the truth of the theory that kept us at work.

In order that you may understand let me illustrate one difficulty. For certain reasons it is necessary that the surfaces of all important molds be waterproofed. This produces, between the wet smooth surface of the cast and the impervious surface of the mold, what the shop men describe as

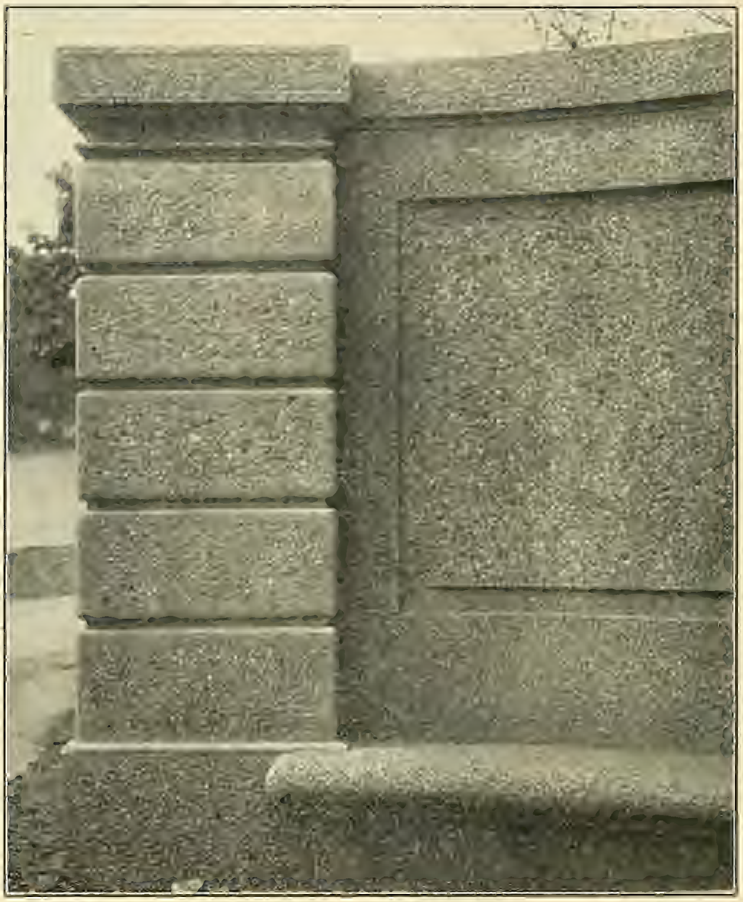

FIG. 3—PIER AND WING WALL AT ENTRANCE TO PARK, SHOWING JUNCTION BETWEEN FIRST AND SECOND OPERATIONS.

Note the decided improvement in surface treatment of the wing wall, seat, and pier cap, as compared with that of the pier. In the latter the matrix predominates, in the former the aggregate.

"suction" and when large surfaces are involved the suction is great. If the casts could be allowed to harden this suction would be of little consequence, as the strength of the concrete would be sufficient to overcome it. But when the concrete is to be surfaced green the mold must be removed at a time when the suction of the mold and the strength of the concrete are about in equilibrium. Consequently instead of the mold coming off clean large spots of concrete stick to it and spoil the cast. I have in mind sometime to continue these papers showing how the molds were prepared, how the concrete was mixed and placed so as to overcome this and other difficulties. Suffice it to say that shop technique has improved to such an extent that a cast is seldom lost. Do not, however, think that all the troubles have been overcome

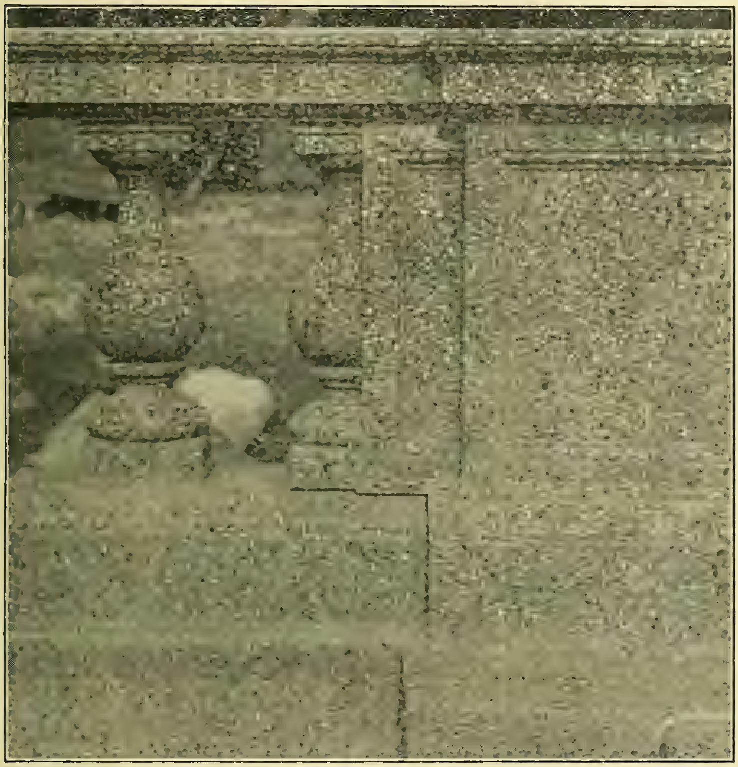

FIG. 4.—ARCHITECTURAL DESIGN OF BALUSTRADE, UNRESTRICTED AS TO FORM.

This concrete has met every requirement in form, color, and texture.

and that these methods are perfected. Quite the contrary is true; the work has just begun.

The first wall was of a cream color obtained by exposing the aggregate; the yellow spots were made to break the surface and the light reflected from them gave a sense of color. It naturally suggested itself that to improve the color a more careful selection of aggregate be made. Great care was taken in looking for and choosing the aggregate, which was finally located in the neighborhood of Washington through a local sand and gravel company whose co operation was a great help.

When the aggregate had been chosen the next most important influence on the color of the work was the frequency with which the spots appeared on the surface. The arbitrary 1 : 2 : 4 mix had to be abandoned and another formula evolved which would deposit the particles of aggregate in the mold as closely to one another as possible. The frequency with which the spots appeared on the surface was principally influenced by the composition and the consistency of the concrete, by the care with which it was placed; by the uniformity with which it was mixed; both as regards the proportioning of the ingredients and the length of time it was allowed to remain in the mixer.

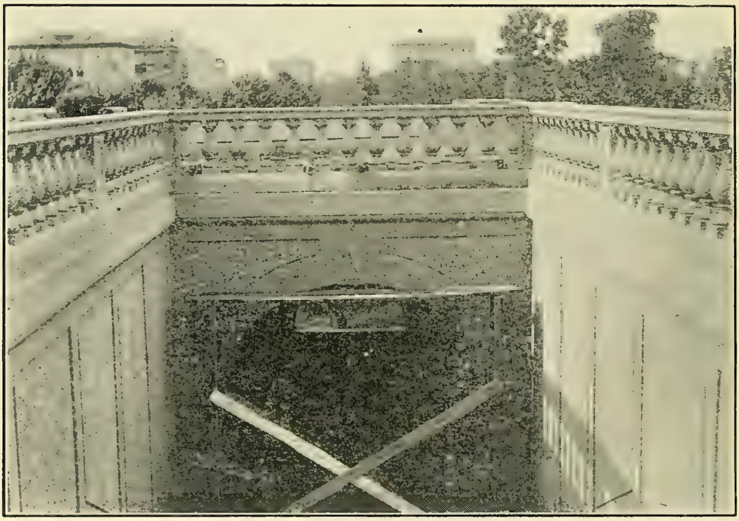

FIG. 5.—VIEW OF OPEN STAIRWAY FROM ABOVE, INDICATING CHARACTER OF THE WORK.

Experience has demonstrated to my satisfaction that the color of architectural concrete (I use the term for want of a better) must be that of a tone obtained by the disposition of small contrasting spots in uniform proportions over the whole surface. Pastelles are a familiar example of this theory of coloring. These surfaces which assume a tone are, upon examination, found to be lines of blue, red, yellow, green, etc. hatched and cross-hatched until they give the effect of uniform coloring. They have an irridescence and a brilliancy which lends to them a charm all their own. And so it is with concrete it is a medium with a charm all its own, which by its nature suggests the pastelle scheme of coloring as opposed to that of solid tints such as are obtained from a coat of paint. I do not think it is possible for satisfactory tints to be obtained in concrete by the addition of pigment to the mortar. I feel that this method is contrary to the nature of the medium. The efforts that I have made in this respect have, without exception, resulted in flat muddy colors.

The third requirement that of texture showed as great an improvement in the second operation as did the color. It was decided that the best way to the desired effect was to abandon the 1 : 2 : 4 mix. The fact that the color and the texture were so closely dependent upon the same method of proportioning greatly simplified the work to be done on the concrete. Many samples were made employing different sizes of aggregate, different size of sand, and different proportions of the ingredients, especially water. Let me impress the fact that, of all ingredients that enter into concrete, none require more exact care in proportioning than the water. Too much or too little water affects the color and the texture through its effect on the arrangement of the pebbles and the case with which the casts are surfaced

When the aggregate is exposed on concrete surfaces the planes and lines should not be irregular and lumpy showing many stones in one place and few in another, but should rather be uniform and true conforming perfectly to the contours of architectural detail. The surfaces should have many uniform particles lying closely together, as the scales on a fish, true and even of surface, yet presenting many irregular planes to the play of light and shade.

My desire is to have you realize that concrete as a plastic medium is capable of expressing architectural detail from the simplest to the most elaborate, is able to meet the most exacting requirements of texture and is subject to beautiful coloring dependent for its range only on our knowledge of aggregates.

An architect friend of mine in lecturing to his class prophesied that structural concrete because of its adaptability to lintel construction and its plasticity would, within a few years, revolutionize building construction and so influence architecture as to found a new school.

Let us not try to find appearance in a concrete into which appearance has not been built. If we do we are doomed to failure. But on the other hand let us build refinement into concrete so that this great material may find a beauty worthy of its strength.

DISCUSSION.

Mr. Wason. MR. L. C. WASON.—I would like to ask Mr. Earley what was the form of matrix used for the forms? Was it lumber or plaster?

Mr. Earley. MR. J. J. EARLEY.—Where the architectural detail is difficult and a few copies are to be made, we have used plaster piece molds. Where there are many repetitions, we have used wooden molds and have waterproofed them. It is necessary that the molds should be waterproofed in order to cast concrete for surface appearance, because wherever there is a fissure in the mold and the concrete escapes, it leaves a seam of cement that surfacing does not remove.

Mr. Wason. MR. L. C. WASON.—What form of waterproofing do you use?

Mr. Earley. MR. J. J. EARLEY.—We use shellac in some cases and in others metal, preferably sheet metal.

Mr. Hasse. MR. O. C. HASSE.—Mr. Earley spoke about a suction which they encountered when removing the form. Did you devise any method of doing away with that so as to strip your molds clean?

Mr. Earley. MR. J. J. EARLEY.—It seemed to us that it was necessary to build up the strength of the concrete at early periods in order to overcome the suction. The best result we had was in the way that we hydrated the cement. We changed the usual order of mixing the concrete. We put the water into the mixture first, and that was followed by the cement, and later by the sand, and later by the aggregate, and the water content was held as low as it was possible to make the mix at all workable. The result was that the strength of the concrete seemed to be much higher at the early periods and did away completely with the scalloping of the surface.

Mr. Chapman. MR. C. M. CHAPMAN.—Did you ever try calcium chloride to hasten the set?

Mr. Earley. MR. J. J. EARLEY.—No, we have never used any addition to the cement, either to accelerate or retard the setting. I have used hot water to accelerate the setting. We have mixed the concrete with boiling water where it was necessary. In one of the views the handrail to the balustrade would be probably 14 or 15 in. across the top and 6 or 7 in. deep, and the length would run from 12 to 14 ft. They were difficult to cast because it was necessary that that should be surfaced on the top and on the bottom and on the side. All four sides had to receive a surface treatment; consequently they had to be removed from the form while they were still green enough to receive the surface treatment, but they also had to be handled in order to receive the finish. We succeeded in making those castings for the handrails so that we could turn them over in 24 hr., surface them, and in 48 hr. carry them around on our overhead trolley to any part of the shop. We did not have any success at all with handling casts of that character until we put into the routine of the shop this method of mixing that I described—the water first, and then

Mr. Earley. the cement and the sand. It is all mixed until it is like a thick soup, and then the aggregate is put in. The water, of course, was predetermined as nearly as it was possible—to such a point that the difference of a pint or a quart of water was all that was necessary in a 5-ft. mix.

Mr. Hatt. MR. W. K. HATT.—Have you given any consideration to the problems which arise after installation, problems of expansion and contraction of the wall which mar the appearance of the wall at the joints?

Mr. Earley. MR. J. J. EARLEY.—Yes, we had to do that because spalling occurs where this expansion is not taken care of and the handrail, of course, was the most aggravated condition we met. It was a continuous line of concrete about 300 ft. long, and it was not restrained in any way, so we frankly let expansion joints at every two lengths; that would be about every 24 ft. We left two $\frac{1}{4}$-in. joints open without pointing of any kind. Where large masses of concrete come together we use expansion felt. It is about $\frac{1}{2}$ in. thick, a bituminous felt such as is manufactured by the Barrett Mfg. Co., and the evidence we received was that it was a very good thing. We did that because in a good many cases it is squeezing out of the joints very considerably.

FIRE TESTS OF CONCRETE COLUMNS.*

By Walter A. Hull.†

The Bureau of Standards is making, at its Pittsburgh laboratories, an investigation of the fire resistive properties of reinforced-concrete columns The need for more definite knowledge than we now have as to the potential ability of different kinds of concrete in different types of construction, to withstand the attack of a severe fire, is perhaps not as generally recognized as it should be. Our concrete structures are built so massively and with such liberal factors of safety, that they usually come through fires of even considerable severity not only without failure but frequently without evidence of any serious injury. Yet, on the other hand, there is an occasional instance of serious structural damage, with great financial loss, in reinforced-concrete buildings by fires of rather short duration and only moderate severity. There is no generally accepted explanation for these exceptional cases in which concrete has not stood up to its reputation and until a satisfactory understanding of the causes of these mysterious failures can be established, concrete structures in general can not be free from the effect, in the public mind, of the uncertainty engendered by these isolated collapses.

This is what may be termed the more flagrant need for further knowledge as to the fire resistive properties of reinforced concrete, and it would perhaps not be premature to point out a more advanced, constructive use for such knowledge. Concrete has been, and is, in this country at least, passing through those pioneer stages in which uncertainties as to the quality of the finished product and also uncertainties in design, are allowed for by rather extravagant factors of safety. This condition is now passing; our knowledge of what can be depended on from a given mixture of concrete, from given materials, is fairly definite and it may be anticipated that greater reliability will result, in design, in the use of higher working stresses for concrete, with a consequent cutting down of the masses in concrete structures. We can not afford to use unnecessary tonnages of materials in the construction of our buildings and one of the absolute essentials to safety in the progress of more effective use of material is intelligent regard for the fire hazard. It is the purpose of the Bureau of Standards, in conducting investigations in fire resistance, to supply additional data that are essential to constructive progress, not only in reducing our fire losses but in doing that without the unnecessary expenditure of resources.

The present investigation was designed to secure, with a minimum number of tests, of full size columns, some definite information as to the effect of the kind of aggregate, the kind of reinforcement and the form of the column, on the behavior of concrete columns under fire, including such data as the temperatures attained in the reinforcement and in the interior of the column, as well as the strength of the column while the highest temperatures existed. In order to accomplish this, it was necessary to confine the number of aggregates to two and to include only two types of reinforcement and two shapes of column. A preliminary investigation, made with smaller specimens, had indicated that among four different aggregates, one each of gravel, trap rock, blast furnace slag and limestone, the grade of gravel under consideration would show the most adverse tendencies and the limestone would be the most free from them. These two aggregates were accordingly selected for the initial column investigation, since they promised to afford strong contrasts in the study of the effect of aggregate. In the matter of reinforcement, two types in common use were selected, namely, 2 per cent vertical steel with 1 per cent of spiral and 2 per cent vertical steel with no spiral. For comparison, columns without reinforcement were included.

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* By permission of the Director of the Bureau of Standards.

† United States Bureau of Standards, Pittsburgh, Pa.

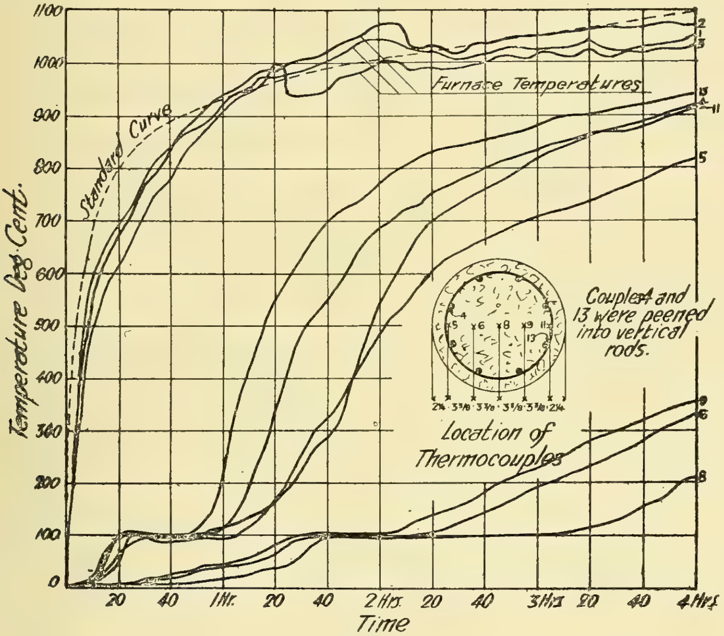


FIG. 1.—COL. 3: 18-IN. GRAVEL CONCRETE COLUMN WITH SPIRAL AND VERTICAL REINFORCEMENT.

Nearly all of the columns were made 18 in. in diameter, if round, or 16 in. square. Round columns, 12 in. in diameter, were made from one aggregate, with one type of reinforcement. In all columns having reinforcement, the latter was placed so as to leave a thickness of 1½ in. of concrete outside the steel. The effective diameter in spirally reinforced columns was taken as the horizontal distance between centers of the hooping. Vertical rods in columns having only vertical reinforcement, were placed at the same distance from the outside of the column as the vertical rods in hooped columns, the effective diameter being assumed to be the same as that for spirally reinforced columns of the same size. In the square columns, the vertical rods were placed at the same distance from the surface of the concrete as in the round ones and the effective area was taken as that portion within a square with its sides located $\frac{5}{32}$ in. beyond the outside of the rods to correspond with the criterion followed in figuring the effective area of the spirally reinforced round columns. All columns were made 8 ft. 9 in. long.

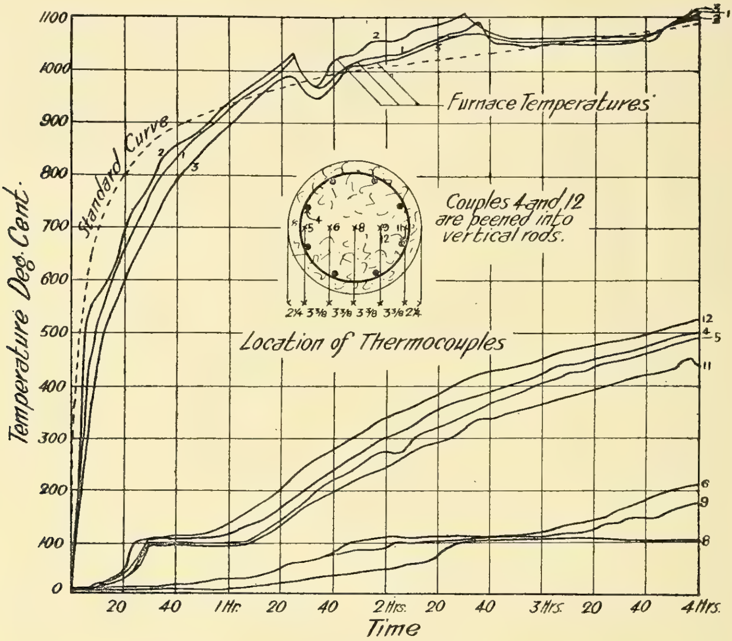


FIG. 2.—COL. 17: 18-IN. LIMESTONE CONCRETE COLUMN WITH SPIRAL AND VERTICAL REINFORCEMENT.

AGGREGATES AND MIXTURE.

The gravel used was the washed river gravel commonly used in the Pittsburgh district for making reinforced concrete. This material is fairly well graded but is somewhat coarse, the largest pebbles being large enough to remain on a 1½-in. screen. In order to remove a variable quantity of fine material, the dry gravel was passed over a screen which passed sizes approximating ¼ in. and smaller.

The limestone aggregate was from quarries at West Winfield, Butler County, Pennsylvania. It is a high calcium limestone. As furnished from the quarries, it was fairly well graded, the sizes ranging up to about ¾ in. In all the columns, the Pittsburgh river sand was used. One portion, 1 : 2 : 4, was maintained throughout the series.

CONSISTENCY.

Concrete was mixed and placed by hand. This was done partly as a matter of convenience and partly because it was considered desirable to use a consistency somewhat less fluid than it is necessary to make with most concrete mixers. It was thought better, for the purpose of the investigation, to use a consistency which could be depended on to give fairly uniform concrete, throughout the investigation, and which would approximate maximum strength for the mixture and materials employed, rather than to attempt to simulate any of the present varieties of commercial practice. The consistency maintained was that resulting from the use of a weight of water of approximately 8 per cent that of the total weight of the dry batch, and it might properly be called a quaking consistency. When first mixed it would stand

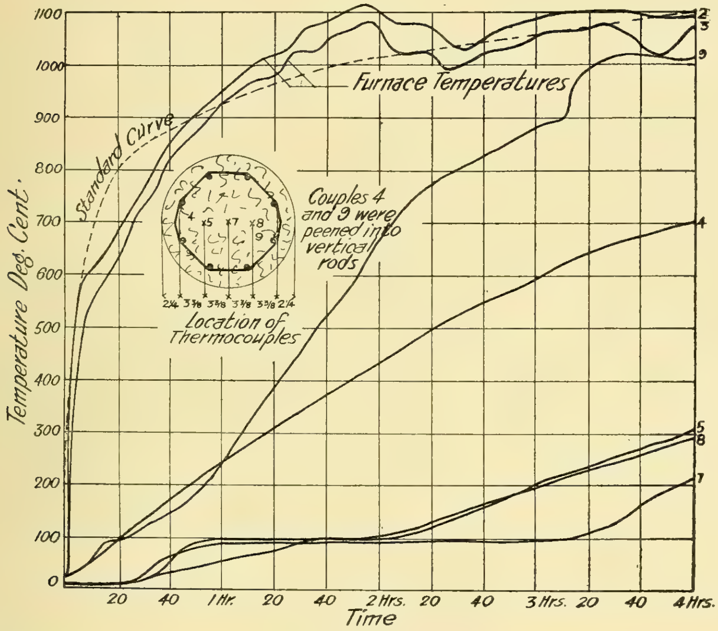


FIG. 3.—COL. 7: 18-IN. GRAVEL CONCRETE COLUMN WITH VERTICAL REINFORCEMENT.

in a slightly flattened mound without spreading out over the floor. The concrete required considerable poling in placing it but it worked well with the pole. There was not much tendency for water to separate and rise to the top; when the form was filled to overflowing, the overflow was mortar, for the most part, rather than separated water. Test cylinders made from the material at the top of the form at the finish of the casting of columns failed to show any consistent difference in strength from test cylinders made from the different batches, before placing. The columns did not show a tendency to give top failures.

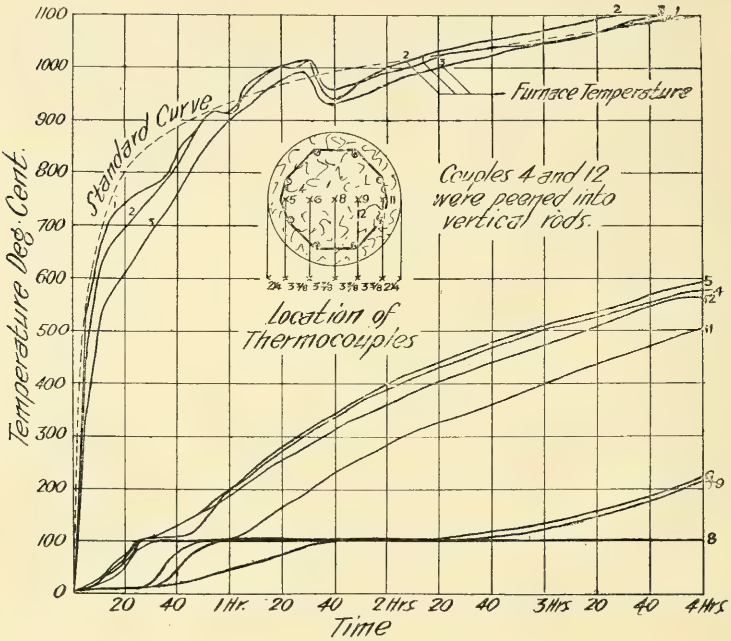


FIG. 4.—COL. 22: 18-IN. LIMESTONE CONCRETE COLUMN WITH VERTICAL REINFORCEMENT.

ORDER OF MAKING AND TESTING.

Three columns of each kind were made up, in order to have, for each type of column, check fire tests, with one cold test, for comparison. In order to avoid the possibility of having all the columns of one type influenced by conditions existing at one time and perhaps not existing when the columns of another type were being made, the rule was followed of proceeding through the list, making, from one type, one column for cold test and one for fire test, then passing on to the next type. After going through the whole list in this way, the third column of the first type was made, and so on, through the series. More than three columns were made in part of the types. In testing, approximately the same order was followed as in making, so that all columns would have been of approximately the same age at the time of test, except that it was not found practicable to accomplish the testing at the same rate as the making. The age of columns ranged, at the time of testing, from six to nine months.

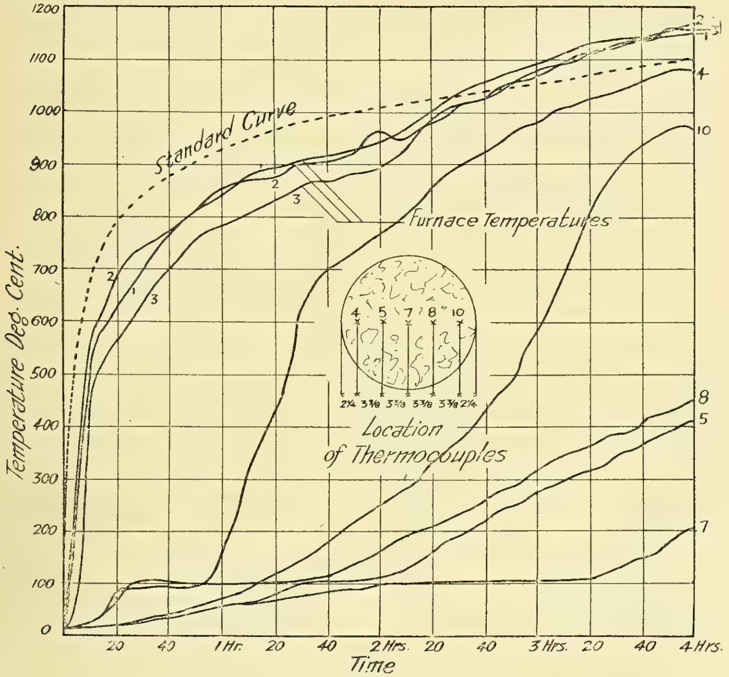


FIG. 5.—COL. 12: 18-IN. GRAVEL CONCRETE COLUMN WITH NO REINFORCEMENT.

TEST APPARATUS.

The test apparatus consisted essentially of a gas fired furnace which is provided with a loading equipment so that a load can be kept on the column during test and that the column can be tested for strength, while still hot, if desired. The furnace is fired with natural gas, using twelve burners of a blast type, compressed air being used to secure a quick and thorough mixture of gas and air, which burns with a short flame. Fires are kept oxidizing. Baffles are so arranged in front of the burners that the flames spread out over the walls of the furnace to some extent on entering the furnace chamber, giving fairly uniform distribution of the heat, throughout the furnace and avoiding any localized impingement on the column.

The load equipment consists of a hydraulic jack, with separate hand pump, the jack being supported above the center of the furnace by two pairs of I-beams which are held by tension rods passing through the ends of a reinforced-concrete girder which extends under the furnace, supporting the foundation for the column. The capacity of the jack is 500 tons. The capacity of the supporting steelwork is rated at 600,000 lb. With the 500-ton jack and hand test pump, the 600,000 lb. load can be applied without difficulty. The jack and test pump, with gage, have been calibrated in one of the testing machines in the engineering testing laboratory of the Bureau, and this calibration is used in the determination of the loads on the columns.

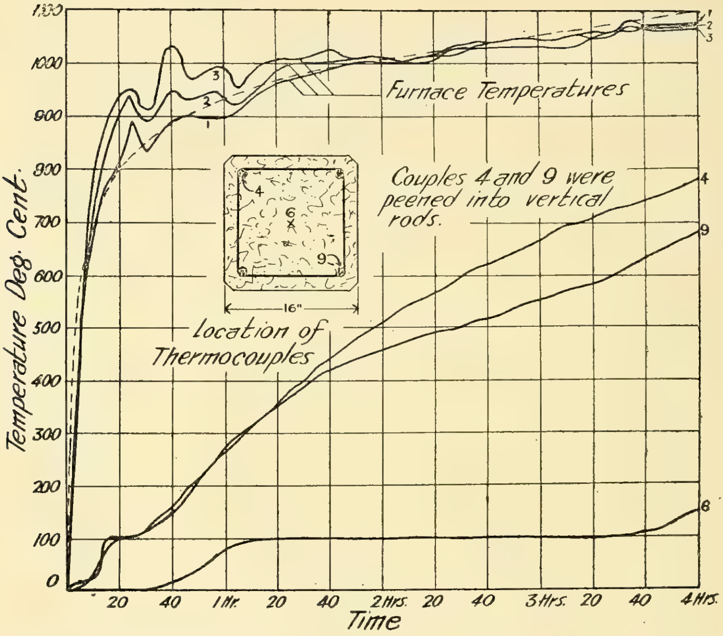


FIG. 6.—COL. 25: 16-IN. SQUARE GRAVEL CONCRETE COLUMN WITH VERTICAL REINFORCEMENT.

Temperatures are measured, both in the furnace and in the interior of the column, by means of iron-constantan thermocouples and a Leeds and Northrup potentiometer indicator. The couples in the column are placed before the column is cast, a special device being used to hold the couples in position during the placing of the concrete.

In all tests, the working load of the column is kept on the column during the fire test. In most of the tests the load has been increased, at the finish of the fire test, up to the ultimate strength of the column or to the capacity of the test apparatus. If the column withstands the 600,000 lb. and load, it is permitted to cool, transferred to the 10,000,000 lb. machine and tested in that.

The firing of the furnace is regulated, as closely as possible, to conformance with the tentative standard fire-test curve, which is shown in Figs. 1 to 8.

TEST DATA ON COLUMNS WITH HOOPING.

That portion of the test data which is calculated to give, in condensed form, the characteristic performance of the different kinds of columns, has been arranged in tabular form, as presented in Tables I to V. It is seen, from an inspection of the tables, that the poorest showing was made by those gravel concrete columns which had both vertical and spiral reinforcement. On the other hand, the best showing was made by columns of the same type but with the limestone aggregate. The reason for this rather remarkable contrast between different columns of the same size and having the same kind of reinforcement, is found in the physical behavior of the different concretes, under fire. With slight variations, the observations made on the gravel concrete columns, Nos. 1, 2 and 3, were as follows: after approximately 30 minutes of firing, cracks began to appear in the surface of the concrete.

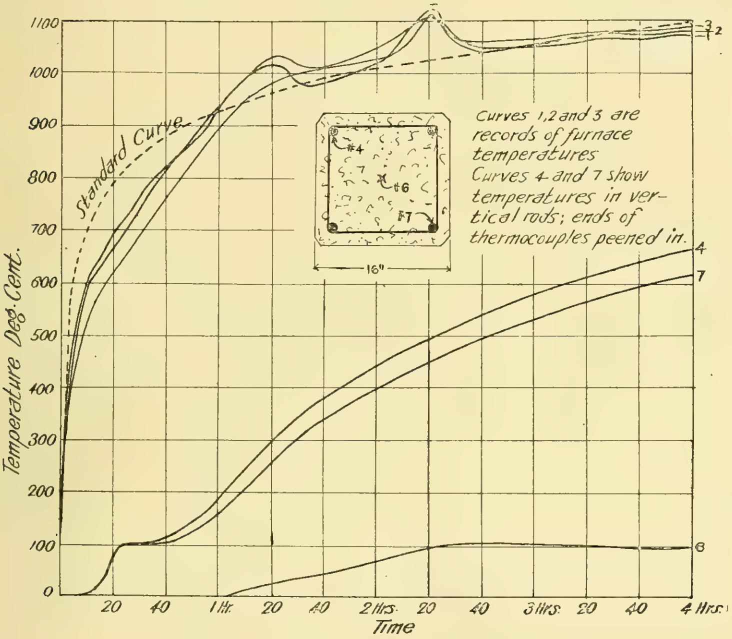


FIG. 7.—COL. 29: 16-IN. SQUARE LIMESTONE CONCRETE COLUMN WITH VERTICAL REINFORCEMENT.

The nature of the cracking was similar to that which is observed in columns of this type when tested cold, just before the final failure. The appearance of the column, at this stage of the fire test, was that of a column that was failing, or about to fail. In reality, it appears to have been actual failure, but failure only of the protective covering, outside the spiral. After this action started its progress was very rapid: cracks lengthened until they intersected other cracks, thus dividing the outer concrete up into large pieces or slabs, which gradually separated from the rest of the column, so that after approximately one hour of firing, slabs would begin to fall, exposing the spiral reinforcement and the load bearing portion of the concrete to the direct heat of the furnace.

TABLE I.

18 in. cylindrical columns.
Thickness of concrete outside the steel, 1½ in.
Reinforcement: 2 per cent vertical, 8 round rods, ¾ in. diam.
1 per cent spiral, $\frac{5}{16}$ in. diam., 2 in. pitch, 2 spacers.
Effective area concrete, 168.7 sq. in.
Area vertical steel, 3.53 sq. in.
Effective area column, 172.2 sq. in.
Working load, 141.500 lb.

	Column Number.	Maximum Load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 Hour Fire Test, deg. C.		
		Without Fire Test.	At End of 4 Hour Fire Test.		In Steel.	Midway between Steel and Center.	At Center of Column.
Gravel concrete.......	1	....	*	....	950	410	240
	2	....	*	....	1050	460	290
	3	....	197,000	1,145	945	355	210
	5	1,090,000	....	6,340	...	...	...
Limestone concrete...	17	....	*	....	520	240	120
	18	....	*	....	560	180	100
	20	1,187,000	....	6,890	...	...	...

* Column No. 1 failed, under working load, at end of 3 hr. 45 min.
Column No. 2 failed, under working load, at end of 4 hr. 15 min.
Column No. 17 loaded, at end of 4 hr. fire test, to 600,000 lb. without failure. Tested later, cold, n large testing machine. Maximum load, 820,000 lb.
Column No. 18 tested in the same way as No. 17. Maximum load, 916,000 lb.

The falling of these broken portions of the outer concrete was a rather slow, gradual process, individual slabs hanging from the top, or standing from the bottom of the column for some time after they had separated to such an extent that the end near the middle of the column stood out some inches from the solid part of the column. However, after one hour of firing, the load bearing portion of the column would be partially exposed and by the end of the second hour a very large portion of the length of the load bearing portion of the column would be completely exposed, so that after the first hour the column proper was only partially protected and after the second hour it had practically no protection at all.

The progress of the temperatures within the column, under these conditions, is seen in Fig. 1. It is evident that at the end of the second hour of firing, the lowest temperature recorded at the depth of the steel was 480° C. and the highest was 770° C. It is known that steel in tension loses 50 per cent of its strength when heated to between 500° C. and 600° C. and it is reasonable to assume that the same is true for steel in compression. It appears, therefore, that in this column, which made the best showing, in every way, of the three columns of this type which were submitted to fire tests, the strength of the reinforcement must have been reduced 50 per cent or more in the first two hours of firing, and that there must have been loss of strength in the load bearing portion of the concrete as well.

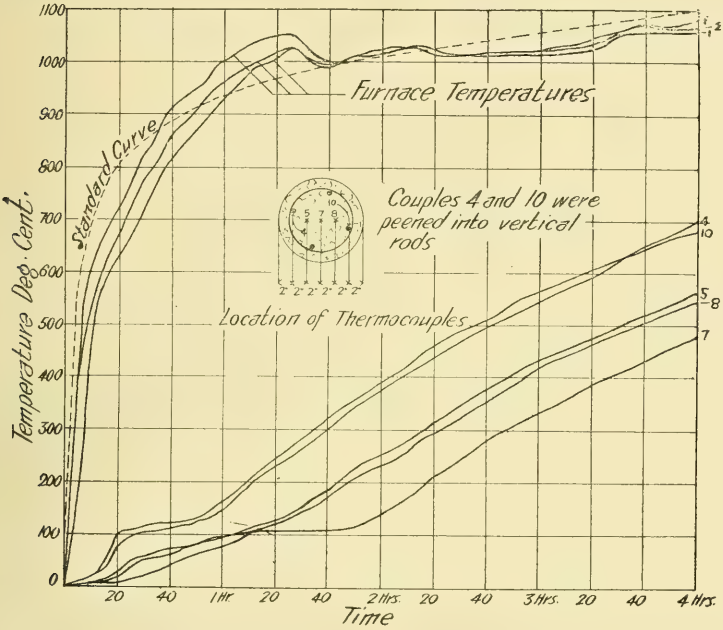


FIG. 8.—COL. 33: 12-IN. GRAVEL CONCRETE COLUMN WITH SPIRAL AND VERTICAL REINFORCEMENTS.

The comparison with the temperatures shown in Fig. 2, for col. 17, is significant. In this spirally and vertically reinforced limestone concrete column, the temperatures recorded at the depth of the steel at the end of two hours, ranged from 240° to 340° C. This difference if attributable, for the most part, to the fact that in the limestone columns, even those with spiral reinforcement, there was very little cracking during the fire tests, the protective concrete staying in place and shielding the load bearing portion of the column as it was designed to do. If we compare the temperature condition of these two columns at the end of the four-hour fire test, it is found that in col. 3, the temperatures recorded at the depth of the steel ranged from 820° to

940° C. and those in col. 17, at the same depth, were between 440° and 530° C. Temperatures at a depth midway between the steel and the center of the column show the same tendency for these two columns, though at this depth the contrasts are naturally somewhat less marked. The effect, however, is seen even at the center of the columns, the temperature at the center in col. 17 being approximately 100° C. as compared with 210° at the center of col. 3. It is evident from the results of the compression tests made after the four-hour fire tests, that in col. 3, not only the steel but the load bearing concrete as well had lost strength very seriously, for the results of the test of col. 15 shown in Table III indicate that the load bearing concrete in col. 3, if uninjured, should have been good for approximately 450,000 lb., without any reinforcement.

TABLE II.

18 in. cylindrical columns.
Thickness of concrete outside the steel, 1½ in.
Reinforcement: 2 per cent vertical, 8 round rods, ¾ in. diam.
Ties, ¼ in. diam., 12 in. centers.
Effective area concrete, 168.7 sq. in.
Area steel, 3.53 sq. in.
Effective area column, 172.2 sq. in.
Working load, 99,750 lb.

	Column Number.	Maximum Load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 Hour Fire Test, deg. C.		
		Without Fire Test.	At End of 4 Hour Fire Test.		In Steel.	Midway between Steel and Center.	At Center of Column.
Gravel concrete.......	7	....	*	....	1,010	310	210
	8	....	235,000	1,365	630	270	130
	10	803,000	....	4,660	...	...	...
Limestone concrete...	21	....	370,000	2,150	580	240	100
	22	....	375,000	2,180	580	230	100
	24	739,000	....	4,290	...	...	...

* Column No. 7, after 5 hour fire test was loaded to failure. Maximum load, 185,000 lb.

COLUMNS WITH VERTICAL REINFORCEMENT ONLY.

Table II shows the same tendencies as Table I but much less marked. In the gravel concrete columns of this type, the protective covering showed a strong tendency to crack in the fire test but shell failure was much less rapid and less complete than in the spirally reinforced columns with the same aggregate. In col. 7, shell failure was evidenced after approximately 30 minutes of firing but the disintegration of the covering proceeded somewhat slowly and the load bearing portion of the column was not exposed to a very great extent by the loss of that portion of the covering which dropped off. In the test col. 8, the rise of temperature of the furnace was somewhat slower than in the other tests on account of low gas pressure and the age of the column was somewhat greater than that of col. 7. The cracking of the outer concrete of this column was much less extensive than in the previous one and the protective concrete stayed in place throughout the test, so that the column had the benefit of practically the full protection of the covering.

It is seen, however, that the showing made by col. 8 is considerably below that of col. 21 and 22, even though the fire test was somewhat less severe in the test of col. 8. The temperatures are slightly higher in col. 8 than in cols. 21 and 22 but this can hardly account for the marked difference in strength and it would appear that the tendency, which has been frequently observed, for concretes from some gravel aggregates to lose strength more seriously, with heat treatment, than concretes from other aggregates, is responsible for the comparatively great loss of strength of this column.

PLAIN COLUMNS.

Table III shows, for comparison, the strength of plain columns, with gravel aggregate. Col. 12 behaved much like the reinforced columns in respect to the breaking up of the outer concrete. Cracking started after about 25 minutes of firing and the outer concrete was well broken up, with large slabs beginning to fall away, by the end of the first hour. This process of stripping the load bearing portion of the column was somewhat less rapid and less complete in the plain column than in the average of those with spiral reinforcement only. The thickness of the portion which split off was somewhat irregular but would approximate 1½ in. or a little more. After the first layer of covering broke loose there did not appear to be any tendency for the same process to repeat itself.

TABLE III.

18 in. cylindrical columns.
Thickness of concrete considered as protective material, 1½ in.
Reinforcement, none.
Effective area, 180 sq. in.
Working load, 81,000 lb.

	Column Number.	Maximum Load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 Hour Fire Test, deg. C.		
		Without Fire Test.	At End of 4 Hour Fire Test.		2¼ in. from Surface.	5⅝ in. from Surface.	At Center of Column.
Gravel.................	12	....	150,000	835	1070	450	205
Concrete...............	15	657,000	....	3,650	....	...	...

SQUARE COLUMNS.

In Table IV it is shown that in the square, vertically reinforced columns the contrast in the behavior of the concretes from the two aggregates is somewhat greater than in the round, vertically reinforced columns of the same total cross-section. In the gravel concrete columns, Nos. 25 and 26, cracking and spalling started in slightly earlier in the burn than in the round columns, slabs splitting off at the corners to the depth of the vertical rods, leaving the rods exposed and the column nearly octagonal in form. The outer concrete in the remaining flat sides cracked and split open extensively and buckled to some extent, but did not fall off. It was not possible to judge as to whether the splits in the sides of the columns extended deeper than the steel ties, but it was found, after failure, that there was a strong tendency for the columns to split up into quarters, above the bottom cone.

In the limestone concrete columns, Nos. 29 and 30, there was no spalling; some fine cracks appeared but at the end of the fire test they were scarcely visible, the protective concrete being, from an insulating standpoint, in good condition.

These results would indicate that vertically reinforced square columns have more of a tendency to spall under fire conditions than round ones with the same sort of reinforcement. The square columns with the limestone aggregate behaved practically the same as the round columns with the same aggregate and the same kind of reinforcement, which indicates that with this aggregate there is not much tendency to spall, in any ordinary form of column.

TABLE IV.

16 in. square columns.
Thickness of concrete outside the steel, 1½ in.
Reinforcement: 2 per cent vertical, 4 round rods, 1 in. diam.
Ties, ¼ in. diam., 12 in. centers.
Effective area concrete, 156 sq. in.
Area steel, 3.14 sq. in.
Effective area column, 159.14 sq. in.
Working load, 92,000 lb.

	Column Number.	Maximum Load, lb.		Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 Hour Fire Test, deg. C.	
		Without Fire Test.	At End of 4 Hour Fire Test.		In Steel.	At Center of Column.
Gravel concrete	25	....	160,000	1,005	775	160
	26	....	132,000	830	995	165
Limestone concrete	29	....	375,000	2,360	660	100
	30	....	385,000	2,420	630	100
	32	810,000	....	5,090	...	...

TWELVE-INCH HOOPED COLUMN.

The data on the one 12-in. column given in Table V would indicate that the showing made by this small column was rather better than that of the 18-in. columns with the same aggregate and the same type of reinforcement. It would be expected, from theoretical considerations, that the reverse would be the case, for it would be expected that the rise of temperature in the interior of the smaller column would be more rapid than in the larger one. The reasno for the failure of the 12-in. column to perform according to theoretical expectations, appears to have been that the protective portion was slower about breaking up and falling off than in the case of the 18-in. columns of the same type. Failure of the protective covering did take place but it broke up slowly and the first slab did not fall away and expose any portion of the steel until about the middle of the fourth hour. This may be attributed to the fact that the thickness of the shell was greater in proportion to the diameter of the column and to the working load, in the small columns than in the larger ones.

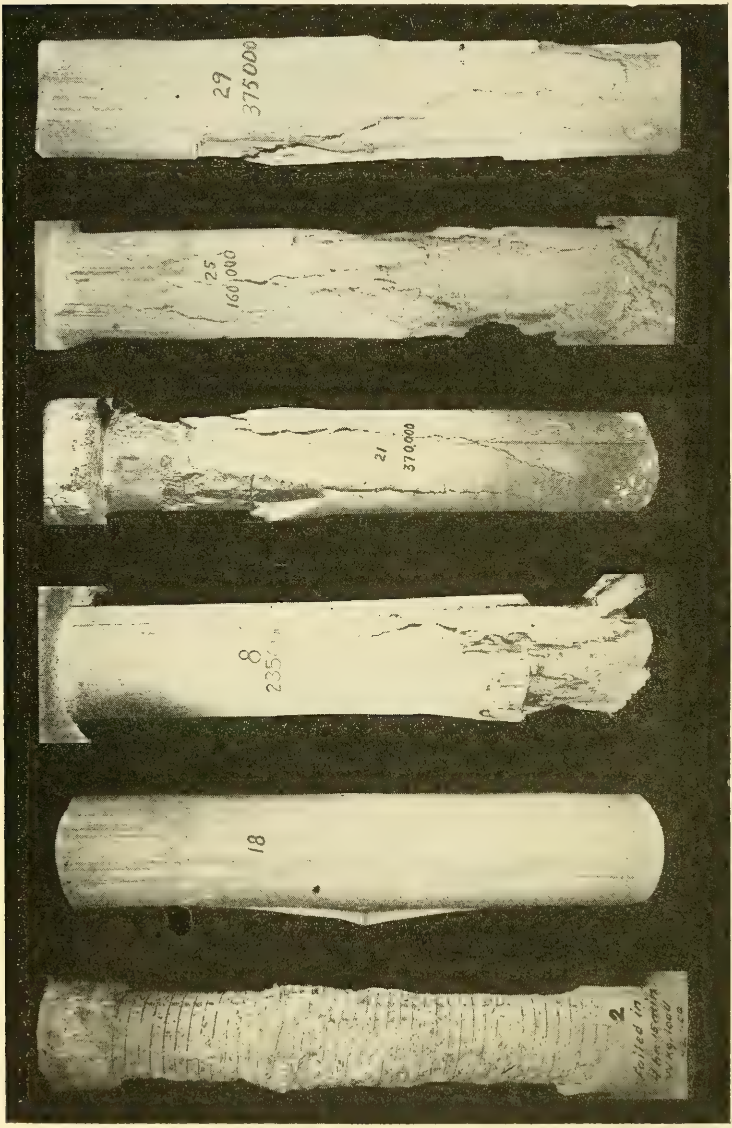
TABLE V.

12 in. cylindrical column.
Thickness of concrete outside the steel, 1½ in.
Reinforcement: 2 per cent vertical, 4 round rods, ⅝ in. diam.
1 per cent spiral, ¼ in. diam., 2⅛ in. pitch, 2 spacers.
Effective area concrete, 60.87 sq. in.
Area vertical steel, 1.23 sq. in.
Effective area column, 62.1 sq. in.
Working load, 50,850 lb.

	Column Number.	Maximum Load at End of 4 Hour Fire Test.	Maximum Stress, lb. per sq. in.	Maximum Temperature at End of 4 Hour Fire Test, deg. C.		
				In Steel.	Midway between Steel and Center.	At Center of Column
Gravel concrete....................	33	71,000	1,145	690	560	480

SUMMARY AND DISCUSSION OF RESULTS.

Gravel Concrete Columns.—It has been observed that in all gravel columns of this series, there has been a marked tendency for the outer concrete to break up early in the fire test, and for the resulting slabs and fragments to separate and fall off, exposing the load bearing portion of the column. The effect of this process, in the most extreme cases, is to strip the load bearing column of its insulation, early in the fire test, with the result that comparatively high temperatures are reached in the steel and in the interior of the column. This spalling is most active in the columns having spiral reinforcement. Round columns with vertical reinforcement, with ties every 12 in., suffered much less from spalling than those with spiral. Square columns, vertically reinforced, spalled and split badly. A round column with no reinforcement showed an intermediate behavior between that of the round columns with spiral and those without. Strength tests made at the end of the 4-in. fire tests showed that gravel concrete columns with spiral reinforcement had retained from less than 15 to approximately 18 per cent of the strength of the duplicate column tested without exposure to heat. A round column with vertical rods and no hooping had retained approximately 30 per cent of its strength, square columns from 16 to 20 per cent and a plain column approximately 23 per cent.



Col. 2.
Gravel concrete failed in 4 hr. 15 min. Concrete came off early in test.

Col. 18.
Limestone concrete. After 600,000 lb. at end of 4 hr. fire test.

Col. 8.
Gravel concrete.

Col. 21.
Limestone concrete

Col. 25.
Gravel concrete. Square.

Col. 29.
Limestone concrete. Square.

FIG 9.—EFFECT OF FIRE IN LOAD TESTS ON DIFFERENT TYPES OF REINFORCED CONCRETE COLUMNS.

These unfavorable results with gravel concrete are similar, in a general way, to those which have been observed, at various times, in laboratory heat tests and in fires in concrete buildings. Prof. Ira H. Woolson, in his final report to the American Society for Testing Materials, in 1907, on work done during 1905, 1906 and 1907, on the effects of heat on concrete, reported adversely on gravel concretes from the pure quartz gravel in extensive use in the vicinity of New York City. The following are extracts from this report:

"The writer is convinced that concrete made from this particular gravel is not reliable as a fire resisting material. Whether other grades of gravel would give equally unsatisfactory results is a matter for investigation.

"The cause for this failure of the quartz mixture is not easy to locate. The most plausible reason seems to be the relatively large coefficient of expansion of quartz. It is about twice that of feldspar, which is one of the predominant minerals of trap rock. Clark's 'Constants of Nature', published by the Smithsonian Institute, gives the cubical coefficient of expansion for these minerals as follows: Quartz, .000036, Feldspar, .000017.

"According to the same authority, quartz has another peculiarity of expansion, namely, that the expansion in the direction of the major axis is only half that in the direction of the axis perpendicular to the major axis. This unequal expansion may further contribute to its tendency to disintegrate the concrete under action of heat."

Referring to this explanation, Professor Woolson says, in his report, published by the National Board of Fire Underwriters, on a fire in a reinforced-concrete warehouse at Far Rockaway, New York, Nov. 10, 1916:

"If this explanation is correct, as evidence thus far produced seems to sanction, all concrete specifications should contain a definite warning against the use of quartz gravel in concrete liable to be exposed to high heat. Where it is so used, Underwriters should take the fact into consideration in assuming the risk."

As already stated, concrete from the Pittsburgh gravel made an unfavorable showing in an investigation recently made by the Bureau of Standards with smaller specimens. The following is taken from the writer's report to the American Society for Testing Materials, in 1917:

"The appearance of the gravel concrete specimens, considered in connection with observations recently made in tests of reinforced concrete columns, indicates that the weakness of the gravel concrete specimens was due, at least in part, to the shattering effect of the stresses caused by the sudden, high expansion of quartz which occurs at the temperature of 575° C., at which the inversion from Alpha to Beta quartz takes place. The magnitude and suddenness of this expansion are shown in Fig. 15,[1] which shows, for comparison, the corresponding volume changes for diabase and for stone concrete. The diabase curve shows the expansion behavior of a material similar to our trap rock and the concrete curve is for 1 : 2 : 5 concrete, the stone probably being trap rock. The quartz and diabase curves are from the work of Day, Sosman and Hostetter,[2] The curve for concrete is from the expansion data of Prof. Chas. L. Norton.[1] The expansion behavior of quartz marks it as a material which would be likely to cause trouble in concretes in which it formed a large part of the aggregate, which is the case in most of our gravels.

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[1] See Fig. 10 of this paper.

[2] "The Determination of Mineral and Rock Densities at High Temperatures. Am. Jour. Sci., Vol. 37, p 1.

There is strong evidence that the peculiar behavior of the gravel concrete in our column tests is due to expansion; yet the Pittsburgh gravel is not made up of pure quartz pebbles. It contains a few quartz pebbles along with a much larger proportion of pebbles of sandstone and of other, harder rock, all appearing to be high in quartz. Inasmuch as most of our gravels have an exceedingly high quartz content, it would seem, from the information available at present, that any gravel aggregate should be investigated before being used, unprotected, in important work requiring fire resistive properties in the concrete.

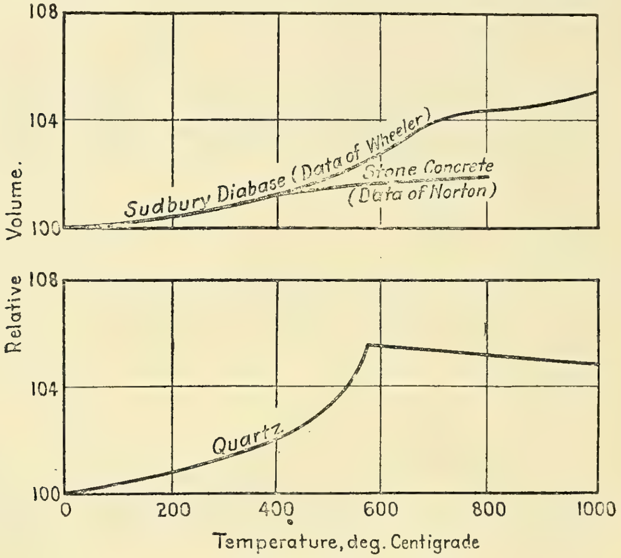

FIG. 10.—VOLUME CHANGES OF QUARTZ, DIABASE AND STONE CONCRETE.

The observations made in the course of these tests seem to indicate that a little anchorage or some light reinforcement, in the protective covering, might serve to hold the latter in place in hooped columns such as these, even though the covering were broken up by expansion. This is to be tried in later tests. It would seem, however, from a careful consideration of the action of these columns, in connection with the peculiar expansion behavior of quartz, that much depends on the suddenness of the heat treatment and that in fires in which the temperature rise is not particularly rapid, the tendency to expansion troubles would be much less than in fires producing heat with great rapidity. It should be taken into consideration that in these fire tests, the columns are not plastered and that the temperature rise is rapid, especially in the first half hour. It is conceivable, and even probable, that the protection of even a rather thin layer of some other material, such as plaster, over the concrete, would be sufficient to make the heating of the concrete enough more gradual to prevent the spalling. Some investigation of this phase of the problem is proposed as a part of the present series. It would appear that gravel concrete of a nature similar to that used in this investigation needs the protection of some other material, for while it is probably true that most fires in buildings do not have as sudden a temperature rise as that of these tests, yet is conceivable that with any one of a number of kinds of occupancy, an exceedingly rapid temperature rise can be produced, in a building, after the first few minutes required for ignition to get well under way, over a large area.

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[1] "Some Thermal Properties of Concrete," Journal, Am. Soc. Mech. Engs., June, 1913.

Limestone Concrete Columns.—In the limestone concrete columns there was no spalling and very little cracking during fire test; this statement applies to square columns as well as to round columns with both types of reinforcement. Consequently, in all the tests of limestone concrete columns, the columns had the full benefit, during the entire fire test, of the insulation of that part of the concrete provided for fire protection and the temperatures shown, both in the steel and in the interior concrete, are correspondingly lower. It is seen, however, that in spite of this protection, temperatures were reached, in the steel and in the load bearing concrete, which would be expected, according to such knowledge as we have, to reduce the strength of both steel and concrete. In the spirally reinforced columns with this aggregate, it is not possible to give the ultimate strength of the columns at the end of the fire test, for obvious reasons, but it evidently was, in both columns, over 50 per cent as great as that of the duplicate column which was tested without fire exposure. These columns, after cooling, showed a strength approximating 70 per cent and 77 per cent, respectively, of the maximum of the unfired column. Each of the vertically reinforced columns, both round and square, without spiral, tested while hot at the end of the fire test, showed a maximum strength approximating 50 per cent of that of the unfired column of the same class.

It is obvious that the comparatively high strength of all the limestone concrete columns, at the end of the fire test, is consistent with the observation that there was no spalling and very little cracking during fire test; for not only did these columns have the full benefit of the heat insulation of the outer concrete, but they must have been comparatively free from expansion stresses of various sorts which must have been important factors in the weakening of the gravel concrete columns. It should also be taken into consideration that there is some advantage, thermally, in the decarbonation of the limestone next to the surface of the column. High calcium limestone decarbonates rapidly, under these conditions, at temperatures approximating 900° C. The reaction is endothermic, the decarbonation of a pound of limestone absorbing approximately the same amount of heat as the evaporation of three-quarters of a pound of water. Heat penetrates concrete with some difficulty and the absorption of heat in the course of its passage through an insulating material may become an important factor in retarding temperature progress in the protected member. This is pretty generally appreciated in connection with the thermal effect of the driving off of water, as in the dehydration of gypsum, but has not been so generally taken into consideration in connection with the driving off of carbon dioxide. Inasmuch as the quantity of limestone in the layer, of a thickness of ⅝ to ¾ in., which was decarbonated in these tests, was considerable, the thermal effect of the heat absorption should not be wholly disregarded. Furthermore, it is probable that because of its greater porosity, the decarbonated material would be a somewhat better heat insulator than the original limestone, so that, from the thermal standpoint at least, decarbonation of limestone may be considered as an advantage. As to compensating disadvantages of the decarbonation, this action is hardly to be charged with any weakening of the column, for any concrete which had attained a temperature of 900° C., could no longer be counted on for any considerable strength. As to salvage, the matter of repair, after a fire, the slaking off of the outer concrete to a depth of say ¾ in. might entail some expense in replacement, provided the concrete did not have to be replaced to a greater depth than this, irrespective of the calcination. But there may be some compensation for this in the fact that the depth to which the decarbonation of the limestone has progressed can serve, at least roughly, as an index to the severity of a fire and the probable condition of the inner concrete; this is at least a better index than judgment based on the depth to which the concrete appears to be dehydrated when attacked with a pocket knife.

Reinforcement.—It has been shown that gravel concrete columns with both vertical and spiral reinforcement gave particularly poor results, even poorer than those which had no hooping; the limestone concrete-hooped columns, on the other hand, made particularly good showings. It is obvious that the steel in the hooped gravel columns could not contribute much strength at the temperatures which it attained; it appears, however, that the hooping had something to do with the rapid breaking up and shedding off of the covering, in these columns, so that the hooping may be regarded, from these results, as increasing the hazard with gravel concrete. With the exception of this one type of column, it may be said that in all cases the reinforcement, although rather thinly covered at the best, gave a fairly good account of itself; it will be observed that in the gravel columns, both the plain ones and those vertically reinforced showed a maximum strength, after fire test, of approximately twice their working load, while both types of limestone columns showed strength approximating four times their working load; in the hooped limestone columns it was evidently somewhat higher than that.

GENERAL CONCLUSIONS.

It is obvious from the results of these tests that there is an important difference in aggregates in respect to the ability of concrete structural members containing them to withstand fire. From the showing made by these gravel concrete columns, together with the other evidence produced, there seems to be no way of avoiding the conclusion that unprotected concrete from some of our gravel aggregates, probably from many of them, is comparatively vulnerable to severe fires in which the initial temperature rise is rapid. This would probably not apply to gravels that were calcareous rather than siliceous in their nature, but would apply to aggregates of crushed stone from siliceous rocks such as sandstones.

The evidence in regard to limestones is less sweeping in its application than that in connection with the gravels because all the evidence presented has been gathered from tests of one kind of limestone. It is probable that some limestones might give better and others poorer results than those produced by this particular grade of stone.

The data so far obtained do not indicate that the proportional credits now being given for reinforcement of the types here represented would be appreciably affected by considering their performance in severe fire tests, provided the protective material remained intact.

It is evident that the insulation afforded by a thickness of 1½ in. of concrete is not sufficient to prevent loss of strength in columns, in a severe fire of long duration. On the other hand, all the limestone concrete columns tested in this investigation have shown an ultimate strength exceeding 3½ times the rated working load of the column. It appears from this that factors of safety employed in present engineering practice are sufficient to allow for the reduction in strength that may be expected in severe fires of four hours duration, assuming that the aggregate is not of a nature which tends to make the concrete liable to spalling. It would appear from these results that if more adequate insulation were provided, the factors of safety used in design could be reduced, so far as considerations of fire hazard are concerned, and that from the fire resistance standpoint, the use of insulating material, other than concrete, in the protection of columns, would be worthy of consideration.

The factors of safety used in engineering practice in the design of load bearing structural members are only such as are considered necessary to assure adequate strength, in these members, at ordinary temperatures. It may be stated, therefore, as a fundamental principle of fire protection, that load bearing structural members, of reinforced concrete or of other materials, should be so protected by heat insulating material that a load bearing member shall suffer no loss of strength by reason of the attainment of higher temperatures in a standard fire test of the duration prescribed for the class of construction for which it was designed.

ADDITIONAL TESTS.

In the interval that has elapsed since the foregoing report was prepared, tests have been made whose results should be given consideration in connection with those already reported, especially such as have a bearing on the protection of gravel concrete.

As has already been stated, the behavior of the protective covering in those gravel concrete columns in which spalling took place, indicated that a little support, as from some light anchorage or reinforcement, would have been sufficient to prevent the broken covering from falling off and exposing the load-bearing portion of the column to the direct heat of the furnace. From theoretical considerations it also seemed probable that if the gravel concrete could have the protection of even a rather thin layer of some additional insulating material, to prevent such rapid heating of the outer concrete as takes place, otherwise, in the standard fire test or in a quick fire, the spalling might be avoided, and the column, protected by both the outer concrete and the additional layer of insulating material, would suffer comparatively little loss of strength. In order to make a trial of such additional protection, two round, vertically and spirally reinforced gravel concrete columns were covered with a thickness of approximately 1 in. of cement plaster, reinforced with a light grade of expanded metal, known as "Floorbinder." The plaster was put on in two coats of the following composition:

First coat: 1 part, by volume, portland cement; 2 parts, by volume, sand; hydrated lime to the amount of 10 per cent (by weight) of the cement. Second coat: Same as above except that hydrated lime to the amount of 15 per cent (by weight) of the cement, was added. No finish coat was applied. The expanded metal used was of the following specifications: Thickness, 16-gage (U. S. Std.)—0.0625 in. Weight, per sheet of 5 ft. by 8 ft. 8 in., 8 lb.

The expanded metal was furred out approximately ½ in. from the column so that it occupied a position at about the middle of the thickness of the plaster.

These two plastered columns were subjected to the four-hour fire test, under working load. The behavior of the plaster, under fire, was not similar to that of the outer concrete in columns of the same type, without plaster. Some cracking took place in the plaster but the cracks were fairly regular, nearly vertical and divided the surface of the plaster into vertical strips. The cracks were first visible after approximately one hour and twenty minutes of firing; they opened, gradually, during the remainder of the test, appearing to be approximately ¼ in. wide, at the surface, at the end of the fire test.

At the end of the fire test both of these columns held the load of 600,000 lb., which is the load limit of the furnace equipment, without failure. One of them, No. 6, has been tested cold, in the 10,000,000-lb. machine, showing an ultimate strength, as shown in Table VI, approximately 10 per cent lower than that of a column of the same type, tested cold, without fire test. The ultimate strength of this plastered gravel concrete column, after fire test, was considerably higher than that of unplastered limestone concrete columns of the same type, and more than four times as great as that of the unplastered gravel concrete columns of the same type. It should be taken into consideration, in this connection, that during a large part of the fire test, the columns of this kind which did not have the plaster did not have any insulating material of any sort outside the steel, because the 1½ in. of protective concrete broke up and commenced to fall off within the first hour. The reinforced plaster served the double purpose of keeping the protective concrete in place and of furnishing additional insulation against heat. The relative strength of plastered and unplastered columns of this sort will be seen to be consistent with the temperatures attained, at the depth of the steel and in the interior of the columns, as given in Tables I and VI.

In order to determine the effect of additional insulation on a square gravel concrete column, one of these, No. 27, was plastered with 1 in. of a gypsum plaster, known as wood-fiber plaster. This material contans no sand but has a wood filler. It was selected for this purpose on the assumption that such a material would give better thermal protection than a gypsum plaster containing sand. This plaster was reinforced with light expanded metal of the same grade and applied in the same way as that used on the round columns referred to in the preceding paragraphs.

TABLE VI.

Column No.	Compare with		Ultimate Strength, lb. per sq. in.		Maximum Temperature at End of 4-Hour Fire Test, deg. C.		
	Table	Column Nos.	At End of 4-Hour Fire Test.	Tested Cold after 4-Hour Fire Test.	At Depth of Steel.	Midway between Steel and Center.	At Center of Column.
6	I	1 to 5	....	5,615	410	160	110
9	I	7 to 10	1,320	....	700	310	170
14	III	12 to 15	....	....	...	...	...
23	II	21 to 24	3,334	....	380	120	90
27	IV	25 to 26	2,880	....	400	...	100
35	V	33	....	....	...	...	...
37	II	....	1,336	....	...	280	130
38	II	....	872	...	...	320	160
41	II	....	1,900	....	530	200	110

NOTES.—Column No. 6 was the same as columns 1 to 5 except that it had 1 in. of reinforced cement plaster over the original column.

Column No. 9 was the same as columns Nos. 7 to 10.

Column No. 14 was the same as columns Nos. 12 to 15. It failed at the end of three hours and forty-five minutes of firing.

Column No. 23 was the same as columns 21 to 24 except that it had, over the original column, 1 in. of a plaster made from a mixture of portland cement and kieselguhr powder, this plaster being reinforced

Column No. 27 was the same as columns Nos. 25 to 26 except that it had 1 in. of reinforced gypsum plaster over the original column.

Column No. 35 was the same as No. 33. It failed at the end of two hours of firing.

Columns Nos. 37 and 38 were the same as those given in Table II except that the aggregate was the so-called Cow Bay gravel and Cow Bay sand from Long Island. This gravel is made up of a mixed lot of pebbles including pure quartz pebbles, some of gneiss and a considerable proportion of granite.

Column No. 41 was the same as columns 7 to 10, Table II, except that the $1\frac{1}{2}$ in. of protective concrete was of cinder concrete from bituminous coal cinders instead of gravel concrete.

It will be seen that the ultimate strength of this column, at the end of the four-hour fire test, was slightly more than three times the average strength of the two columns of the same kind which had no plaster. In this case, as in the preceding one, a comparison of the temperatures attained in the plastered and in the unplastered columns shows the important thermal effect of the additional insulation. In the unplastered square columns, a relatively small part of the protective covering spalled off in the fire test and the lower temperature attained at the center of the plastered column is to be credited largely to the thermal insulation of the plaster.

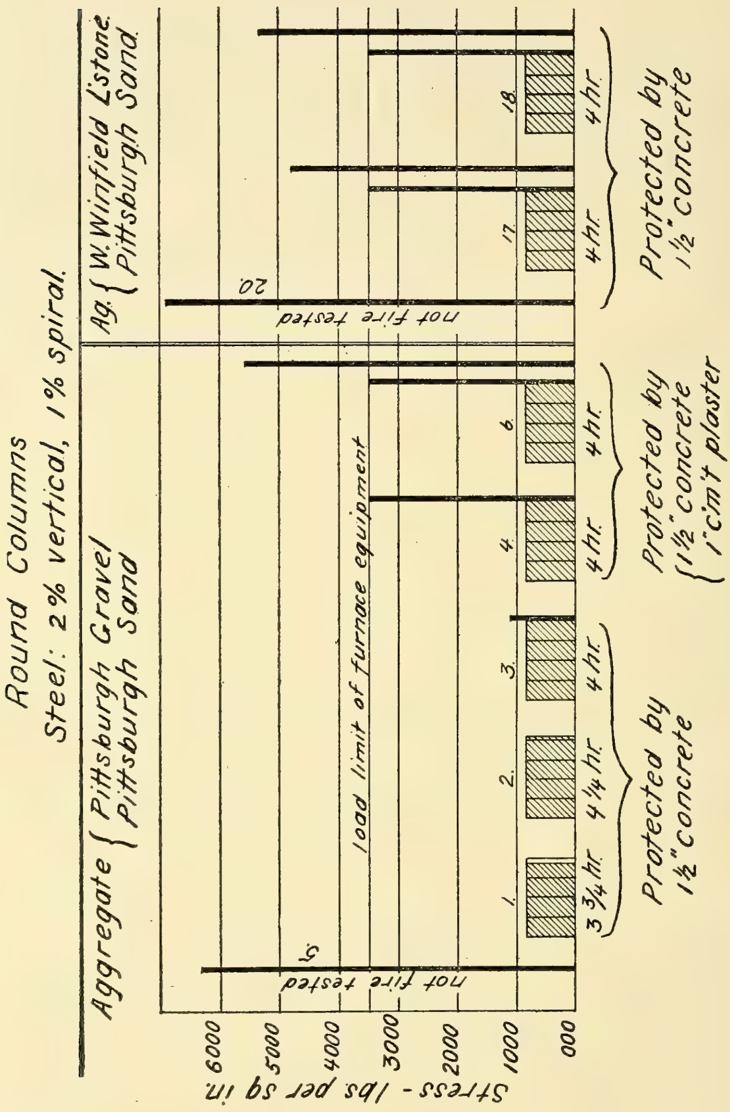
Round Columns
Steel: 2% vertical, 1% spiral.
Aggregate { Pittsburgh Gravel
Pittsburgh Sand
Ag. { W. Winfield L'stone
Pittsburgh Sand
not fire tested
5.
1.
2.
3.
4.
6.
not fire tested
20.
17.
18.
load limit of furnace equipment
3 3/4 hr.
4 1/4 hr.
4 hr.
4 hr.
4 hr.
4 hr.
4 hr.
4 hr.
Protected by
1½" concrete
Protected by
{ 1½" concrete
1" c'm't plaster
Protected by
1½" concrete
6000
5000
4000
3000
2000
1000
000
Stress - lbs. per sq. in.

FIG. 11.

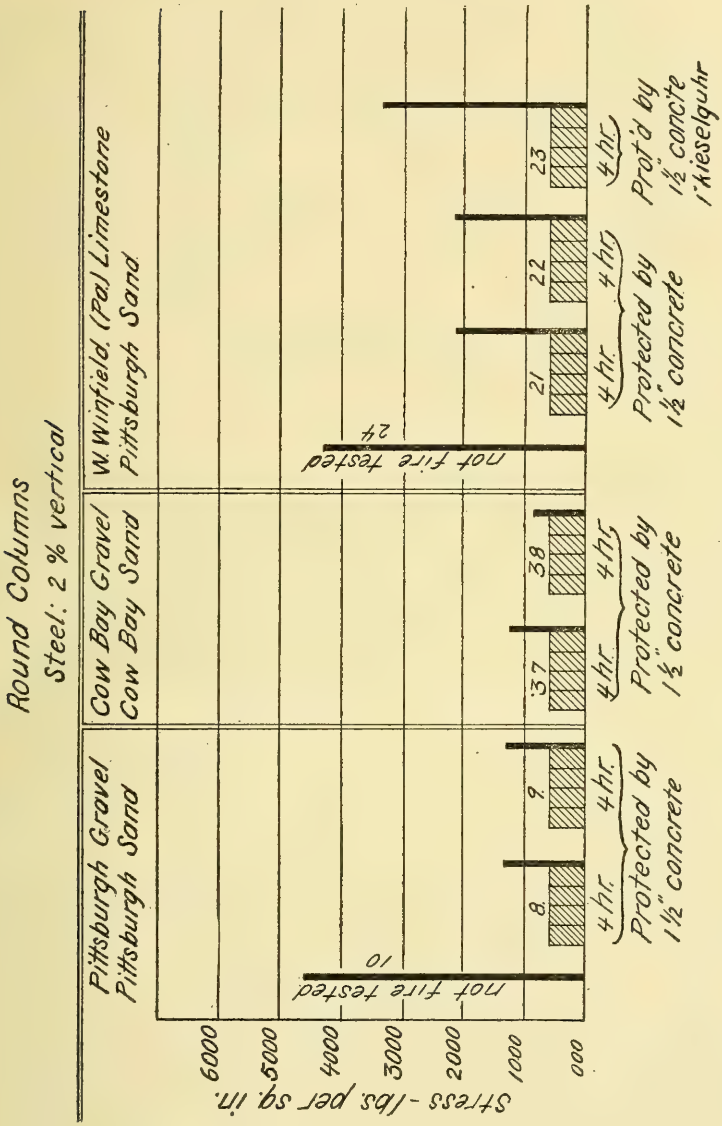
Round Columns
Steel: 2 % vertical
Pittsburgh Gravel
Pittsburgh Sand
Cow Bay Gravel
Cow Bay Sand
W. Winfield, (Pa.) Limestone
Pittsburgh Sand
Stress - lbs. per sq. in.
6000
5000
4000
3000
2000
1000
000
not fire tested
10
8
9
4 hr.
4 hr.
Protected by
1½" concrete
37
38
4 hr.
4 hr.
Protected by
1½" concrete
not fire tested
24
21
22
23
4 hr.
4 hr.
4 hr.
Protected by
1½" concrete
Prot'd by
1½" conc'te
1" kieselguhr

FIG. 12.

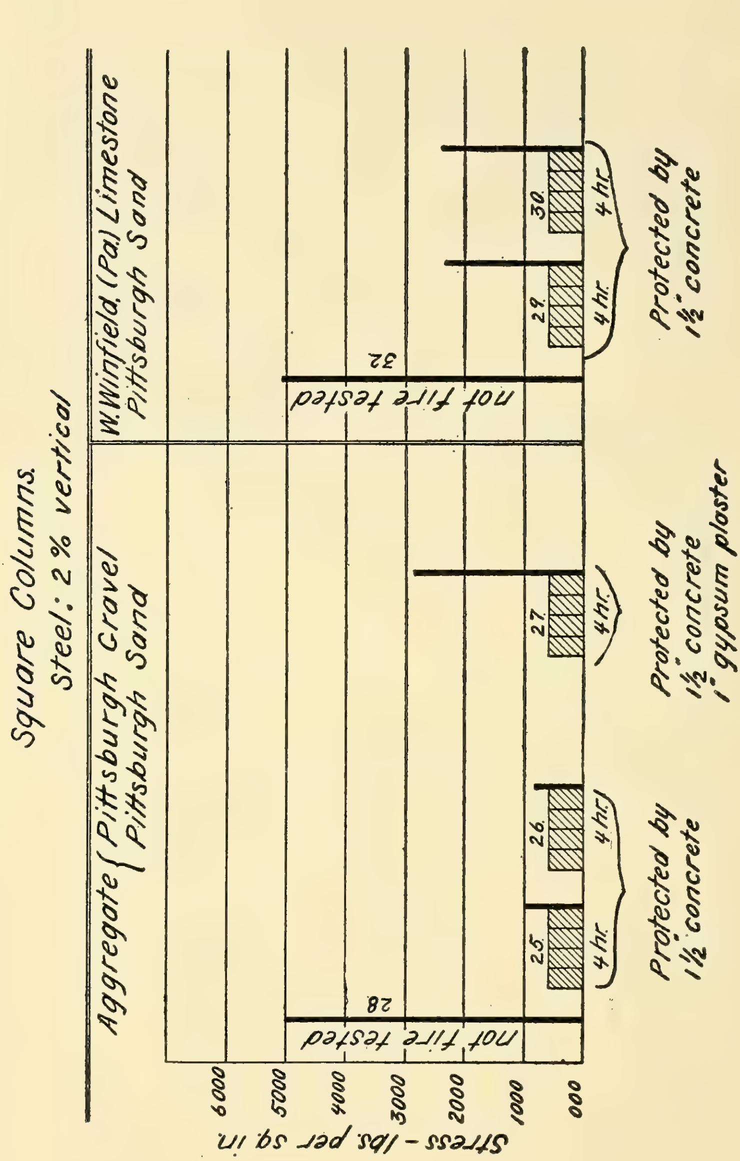
Square Columns.
Steel: 2% vertical
Aggregate { Pittsburgh Gravel
Pittsburgh Sand
W. Winfield, (Pa.) Limestone
Pittsburgh Sand
Stress - lbs. per sq. in.
6000
5000
4000
3000
2000
1000
000
not fire tested
28
25.
4 hr.
26.
4 hr.
Protected by
1½" concrete
27.
4 hr.
Protected by
1½" concrete
1" gypsum plaster
not fire tested
32.
29.
4 hr.
30.
4 hr.
Protected by
1½" concrete

FIG. 13.

It seems reasonable to assume that the effect of fire on concrete structural members is not confined to the loss of strength in the steel and concrete due to changes in the properties of these materials resulting from increase of temperature. It is probable that expansion strains caused by the unequal heating of different portions of a member may play an important part in the reduction in strength. There can be little doubt that such strains would be relatively great in gravel concrete members because of the peculiar expansion behavior of quartz referred to earlier in this report; they would unquestionably be much more severe in the case of rapidly heated members than in those heated more slowly. It is therefore to be concluded that in these gravel concrete columns which had the protection of additional insulating material, the comparatively small reduction in strength is to be credited in part to the absence of excessive expansion strains.

The effect of additional protection on a limestone concrete column is shown in the data given for Column No. 23. This round, vertically reinforced limestone concrete column was covered with 1 in. of a plaster made from a mixture of one part portland cement to three parts by volume of kieselguhr powder, the plaster being reinforced in the same manner as those already referred to. This mixture, even with the addition of fiber, cracks badly in drying and is not to be regarded as a practical combination for commercial use. The results of the test are given to show the effect of additional insulation on the fire resistance of a limestone concrete column. It is to be taken into consideration that columns Nos. 21 and 22 came through the fire test without the loss of any of their protective concrete and without any important cracks, so that the difference between the temperature condition of these columns and that of column No. 23, at the end of the four-hour fire test, is to be attributed to the additional insulation. Furthermore, the strength of column No. 23, at the end of the fire test, may be regarded as consistent with its temperature condition.

The results of the tests of a number of columns are shown graphically in Figs. 11 to 13. In these charts, the four-hour fire test is indicated by the shaded area and the strength of the column immediately after the end of the fire test is shown by the height of the heavy vertical line attached to the shaded area. Those vertical lines which terminate at the load limit of the furnace equipment, 600,000 lb., indicate that the corresponding columns did not fail, at the end of the fire test, under the 600,000-lb. load. Heavy vertical lines to the right of lines which terminate at the load limit give the strength of the corresponding columns when tested in the 10,000,000-lb. machine, some time after the fire test.

It would perhaps be premature to attempt to draw any conclusions from the results of these tests of insulated columns, except such as have been indicated in the preceding paragraphs. It is clearly indicated, however, that a relatively small thickness of insulating material can afford, to concrete columns, relatively great protection against a severe fire. This is particularly true in the case of gravel concrete. Further investigation will of course be necessary to the proper development of the application of these findings to structural problems.

MOMENT COEFFICIENTS FOR FLAT-SLAB DESIGN WITH RESULTS OF A TEST.

By W. K. Hatt.*

The analysis of the mechanical action of a thin flat plate composed of a material like steel, involving elastic constants, and inter-action of stresses on right-angled axes, has exercised the talents of famous writers on mechanics. The same analysis and development have been carried over by a group of writers into the treatment of the reinforced-concrete flat slab, a comparatively thick, partially elastic slab, supported on columns with enlarged heads that effectively reduce the working span. In carrying over the treatment, of a thin steel plate supported on points, to a reinforced-concrete flat slab, it is necessary to assume properties of elasticity that are not possessed by the material, and inter-actions of component parts that, as it appears to the writer, are pure assumptions. Recent discussion has arrived at widely different results as to the moments to be provided for in design.

A second group of writers has treated the reinforced-concrete flat slab as a wide beam, according to the principles of simple flexure, and has inquired what the moments are when the decrease of span due to the wide column heads is taken into account.

The first group of writers endeavor to check the results of its analysis by reference to the resisting moments developed in tests of buildings of flat-slab construction, where steel stresses have been measured. From these steel stresses the moment of resistance of the slab has been computed, and this moment is then compared to the moment resulting from the analysis.

Of course, this process is fallacious because only one element of the total moment is considered. Actually the total resisting moment is made up of two parts: (1) that part of the moment supplied by the steel stresses and (2) that part supplied by the tensional stresses in the concrete.

No check of theory by test can be made on such a basis.

The author desires to examine existing tests on reinforced concrete to determine the relation between the total external moment and the moment resident in the steel stresses in the case of:

(1) Simple Beams in the Laboratory.
(2) Continuous Beams in Building Tests.
(3) Flat-Slab Building Tests.
(4) Flat-Slab Test Panels.

It will appear that there is little difference in the behavior of flat slabs and simple beams in respect to this matter under examination. The moment coefficient appearing in steel stresses at a load is a question of the quality of the concrete, as well as of the amount of steel.

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Of course it would be desirable to determine moments of resistance on the basis of compressive concrete stresses, but it is difficult to translate deformations of concrete into terms of stress on account of the uncertainty of modulus of elasticity in this semi-plastic material, and the possibly different modulus in concrete in structures as compared to cylinders under the action of a testing machine.

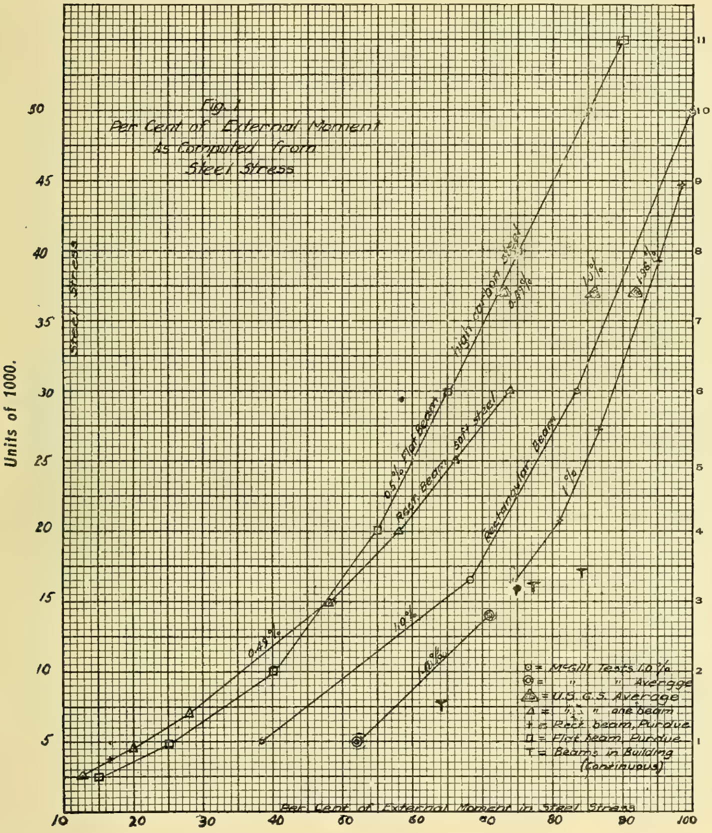


FIG. 1.—PER CENT OF EXTERNAL MOMENT ACCOUNTED FOR BY STEEL STRESS—BEAMS.

SIMPLE BEAMS IN THE LABORATORY.

The author, as early as 1902, recognized the importance of the rôle played by tensile stresses in concrete in the action of the entire resisting moment of the beam.

Some examples of carefully conducted tests, where external moment may be compared with moment developed by steel stresses, may be quoted as follows:

(1) Tests by Professor E. Brown, of McGill University (Trans. Can. Soc. C. E., Vol. 22, p. 245).

Beams loaded at third points; neutral axis determined by extensometers on vertical faces of beam; deformations of steel inferred from surface measurements. One per cent steel approximately.

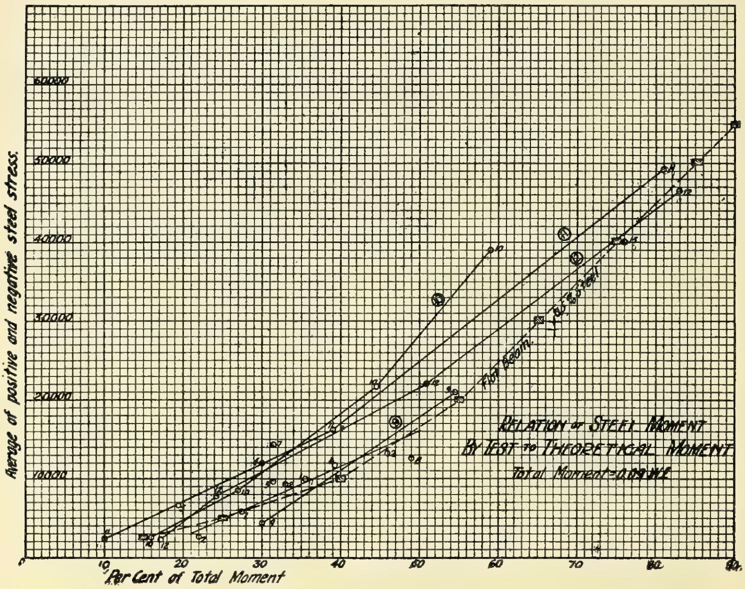


FIG. 2.—PER CENT OF EXTERNAL MOMENT ACCOUNTED FOR BY STEEL STRESS—TOTAL MOMENT, BUILDINGS.

For example, one beam yielded the following:

UNIT STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF LOAD MOMENT IN STEEL STRESSES.
4,800	37.8
16,500	68.5
30,000	83.5
50,000	100.0

Eleven such tests at McGill University showed an average per cent of external moment to be resident in steel stresses as follows:

STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF EXTERNAL MOMENT.
5,000	52
14,000	71

(2) Tests by Richard L. Humphrey and Louis H. Losse, published in Bureau of Standards Technologic Papers No. 2, 1912.

Deformations of steel inferred from deformations of concrete on face of beam. Neutral axis determined. Beams rectangular, loaded at 1/3 points.

Those of age of 4 weeks reinforced with from 0.49 to 1.96 per cent of steel, show the following at stresses from 30,000 to 44,000 lb. per sq. in.

PER CENT OF REINFORCEMENT.	PER CENT OF EXTERNAL MOMENT.
0.49	73
0.75	84
1.00	86
1.50	90
1.96	92

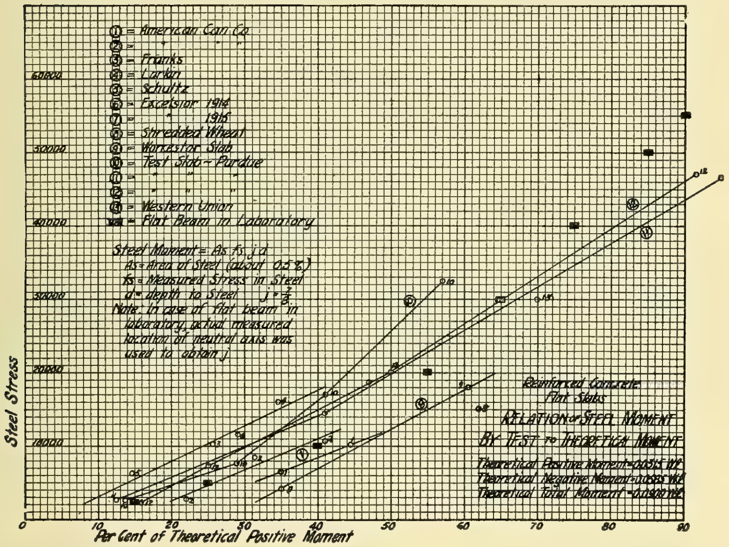


FIG. 3.—PER CENT OF EXTERNAL MOMENT ACCOUNTED FOR BY STEEL STRESS—POSITIVE MOMENT, BUILDINGS.

Evidently as the amount of steel decreases the proportion of the moment supplied by the tension in the concrete increases.

Selecting a beam with 0.49 per cent reinforcement to examine the behavior during the test, we find the following:

Beam reinforced with 0.49 per cent steel.

STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF EXTERNAL MOMENT.
2,500	12.5
4,500	20.0
7,000	28.0
15,000	48.0
20,000	58.0
25,000	66.0
30,000	74.0

(3) A rectangular beam with 0.90 per cent reinforcement tested in Purdue University Laboratory.

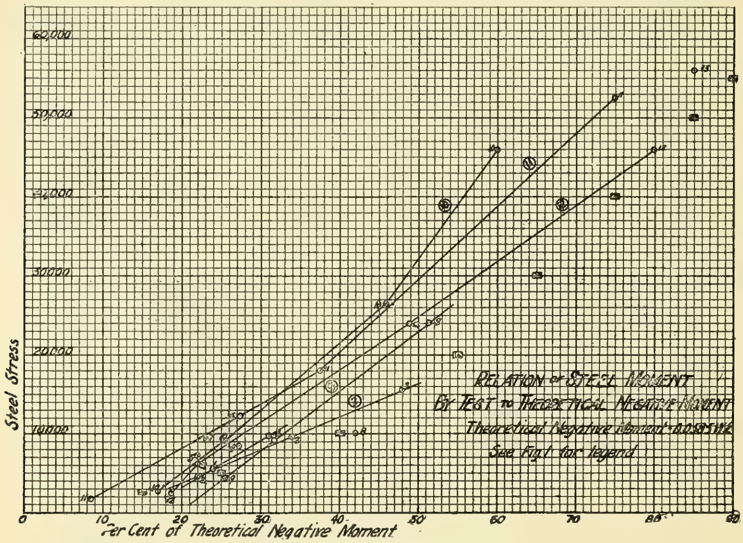
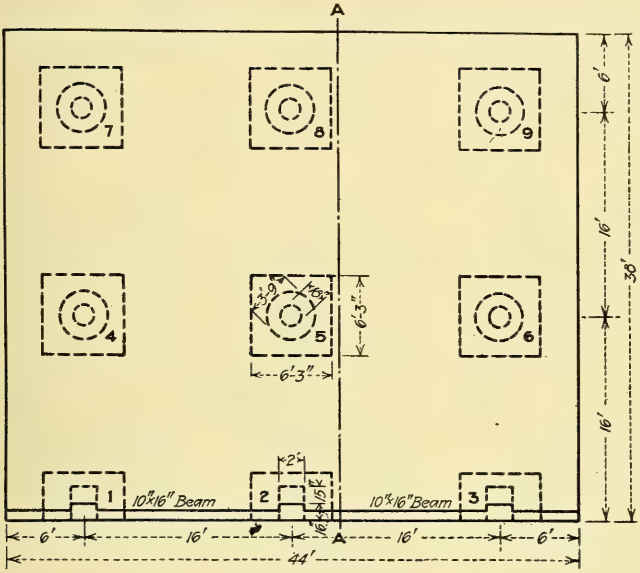


FIG. 4.—PER CENT OF EXTERNAL MOMENT ACCOUNTED FOR BY STEEL STRESS—NEGATIVE MOMENT, BUILDINGS.

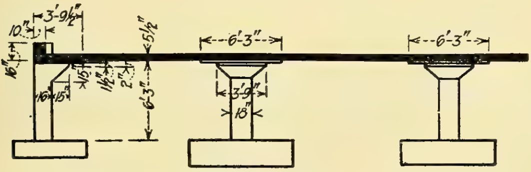
STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF EXTERNAL MOMENT
2,600	16.5
16,300	74.4
21,800	81.5
27,200	86.5
39,300	95.5
44,700	98.1

(4) A flat beam 27¼ in. wide, 5.54 in. deep, with 0.5 per cent of steel, and tested at Purdue University under a third-point loading on a 75-in. span, yielded the following:

STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF EXTERNAL MOMENT.
2,500	15
5,000	25
10,000	40
20,000	55
30,000	65
40,000	75
50,000	85
55,000 yield point	90



PLAN OF FLOOR



SECTION A-A

FIG. 5.—CONCRETE DIMENSIONS, J SLAB.

The various tests have been plotted in Fig. 1.

The following relations hold for these tests:

(1) That apparently lower coefficients of resisting moment as determined from steel stresses, result when the per cent of steel decreases.

(2) That the per cent of moment and the apparent moment coefficients become greater as the stress in the steel rises during the test, and the concrete breaks down in tension.

(3) At measured stresses in the steel of 10,000 lb. per sq. in., there is nearly 60 per cent of the total moment provided by tensile stresses in the concrete for the beams with 0.5 per cent of steel, and about 40 per cent in beams with 1.0 per cent of steel.

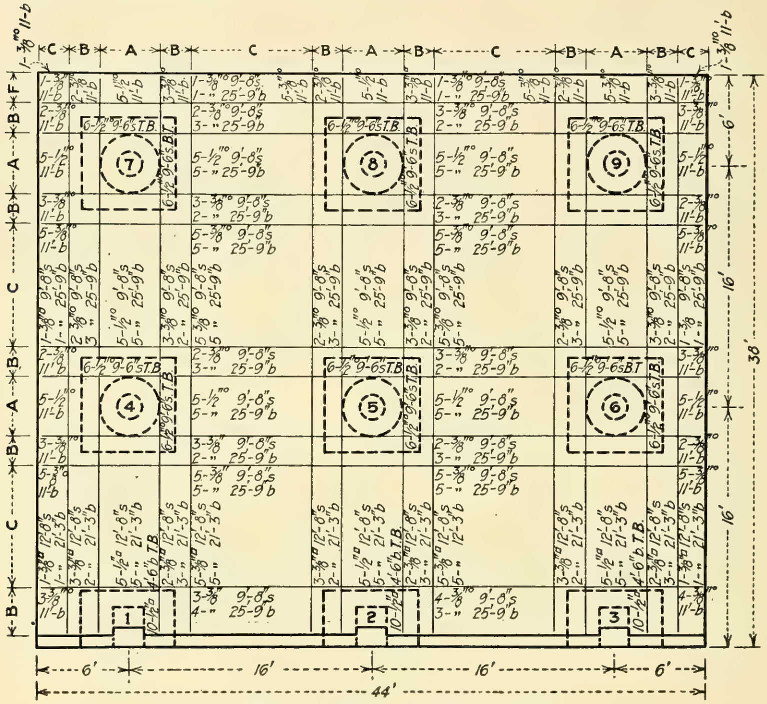


FIG. 6.—STEEL LAYOUT, J SLAB.

These tests are examples, but so far as the author knows the computations from similar tests will bear out the truth of these relations, which will be modified in degree by various qualities of concrete, and time-rate of loading.

TESTS OF BEAMS IN BUILDINGS.

Tests of buildings whether of beam and girder type or of flat-slab type, are imperfect measures of the relation sought.

(1) The floor is only partially loaded, and there are contributions to the resisting moment arising from columns and unloaded areas, exterior to the loaded portion of the floor. There are also vacant spaces left for observation.

(2) The position of the steel bars is only known in part.

(3) The time rate of application of load, and the temperature conditions, which play an important rôle in measured deformations, are not well controlled.

(4) Thickness of floor finish is uncertain.

(5) Comparatively few locations for measurements are chosen.

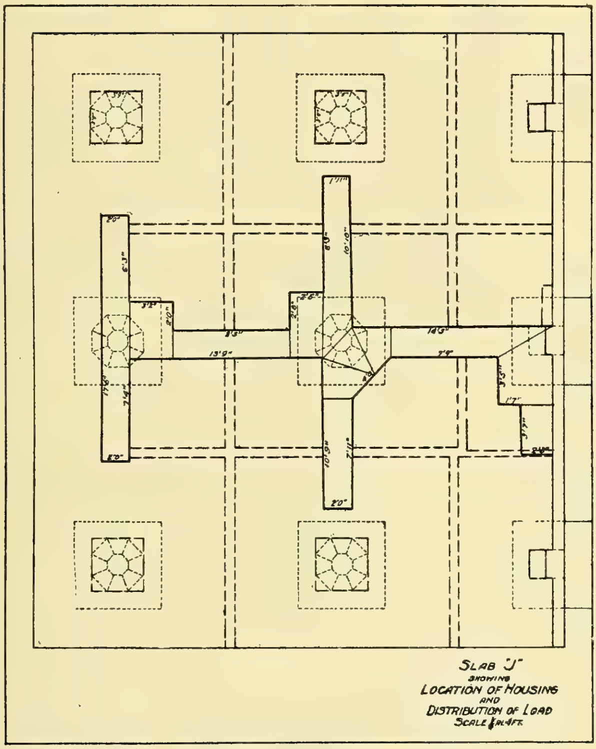


FIG. 7.—LOADING DIAGRAM, J SLAB.

(6) Sometimes there is arching action in the loading material.

(7) Errors of measuring deflections creep in from the attempt to use an engineer's level, or to measure deflections from floor below.

In computing the resisting moment in case of buildings, the value of the leverage of the steel has been taken as ⅞ *d*. This will be less accurate at low steel stresses, than at high steel stresses.

The resisting moment of certain beams in buildings are calculated in Bulletin 84 of Engineering Experiment Station of University of Illinois as follows:

	STRESS IN STEEL, LB. PER SQ. IN.	PER CENT OF EXTERNAL MOMENT.
Continuous Beam	16,000	77.0
Continuous Beam	7,500	64.0
Continuous Girder	17,000	79.0

These have also been recorded on Fig. 1. These carry more moment in the steel than do simple beams in the laboratory.

TABLE I.—DATA OF MOMENT COEFFICIENTS FROM STEEL STRESS TESTS OF FLAT SLABS, JUNE 1, 1918.

	Positive Stress.	Positive Coefficient.	Per Cent of 0.0315	Negative Stress.	Negative Coefficient.	Per Cent of 0.0585	Total Coefficient.	Per Cent of 0.090
1. Sanitary Can Bldg. (4 panel)	2,700	......	....	.....	......	....	......	....
	6,350	0.011	35.0	5,500	0.014	24.0	0.025	27.6
	10,150	0.014	44.5	9,880	0.018	31.0	0.032	35.5
2. Shonk Bldg. (4 panel)	2,800	0.007	22.2	2,450	0.013	22.0	0.020	22.0
	8,550	0.010	31.5	9,650	0.020	34.0	0.030	33.0
	10,800	0.013	41.0	15,570	0.028	48.0	0.041	46.0
3. Franks (2 panel)	10,100	0.0081	25.7	9,200	0.0185	31.6	0.0286	31.5
4. Larkin	16,000	0.0109	34.8	8,500	0.0161	27.2	0.0270	30.0
5. Schultz	6,200	0.0046	14.6	7,100	0.0129	22.0	0.0175	19.5
6. Excelsior, 1914	11,600	0.0092	29.2	12,300	0.0161	27.2	0.0263	39.2
7. Excelsior, 1915	18,900	0.0148	47.0	9,300	0.0134	23.0	0.0282	31.5
8. Shredded Wheat (9 panel)	15,000	0.0195	62.0	10,000	0.0244	42.0	0.0439	49.0
9. Worcester Slab	4,100	0.011	35.0	4,200	0.015	25.6	0.027	30.0
	18,000	0.019	60.5	24,000	0.030	51.2	0.049	54.5
10. Slab J	2,048	0.0048	13.8	3,000	0.010	17.0	0.0144	16.0
	7,970	0.0095	29.0	9,000	0.015	25.6	0.0242	27.0
	17,000	0.0130	41.0	26,500	0.027	46.0	0.0400	44.5
	32,500	0.0184	57.0	46,000	0.035	60.0	0.0540	59.0
11. Slab S	2,400	0.0043	12.6	1,870	0.0056	8.5	0.0099	10.0
	14,200	0.013	41.0	18,000	0.0228	37.5	0.0358	39.0
	46,700	0.029	95.0	52,500	0.0442	75.0	0.0732	81.0
12. Slab R	2,200	0.0055	16.0	2,700	0.0115	18.8	0.017	17.8
	7,200	0.0083	25.0	6,200	0.0138	22.0	0.0221	24.2
	20,400	0.0165	50.0	24,000	0.0298	49.0	0.0463	51.0
	47,500	0.0283	92.0	47,000	0.0463	80.0	0.0746	83.0

TESTS OF FLAT SLABS IN BUILDINGS.

A number of flat-slab buildings have been tested during the past five years, and low stresses found. These tests are to be interpreted in the light of the remarks made above. Those on record are quoted below. The coefficients are as found in the various published reports, in the Bulletin 84 of the Engineering Experiment Station of the University of Illinois and in published reports by A. R. Lord and the author. To these are added certain tests recently made at Maywood, Ill.

In diagraming these results the positive moment at panel centers, the negative and positive moment, and the total moment are all used for a criterion. The positive moment is taken to be 35 per cent of the total amount of 0.09 *Wf*, which is distributed, as shown by the results of tests of panels, into 65 per cent negative and 35 per cent positive.

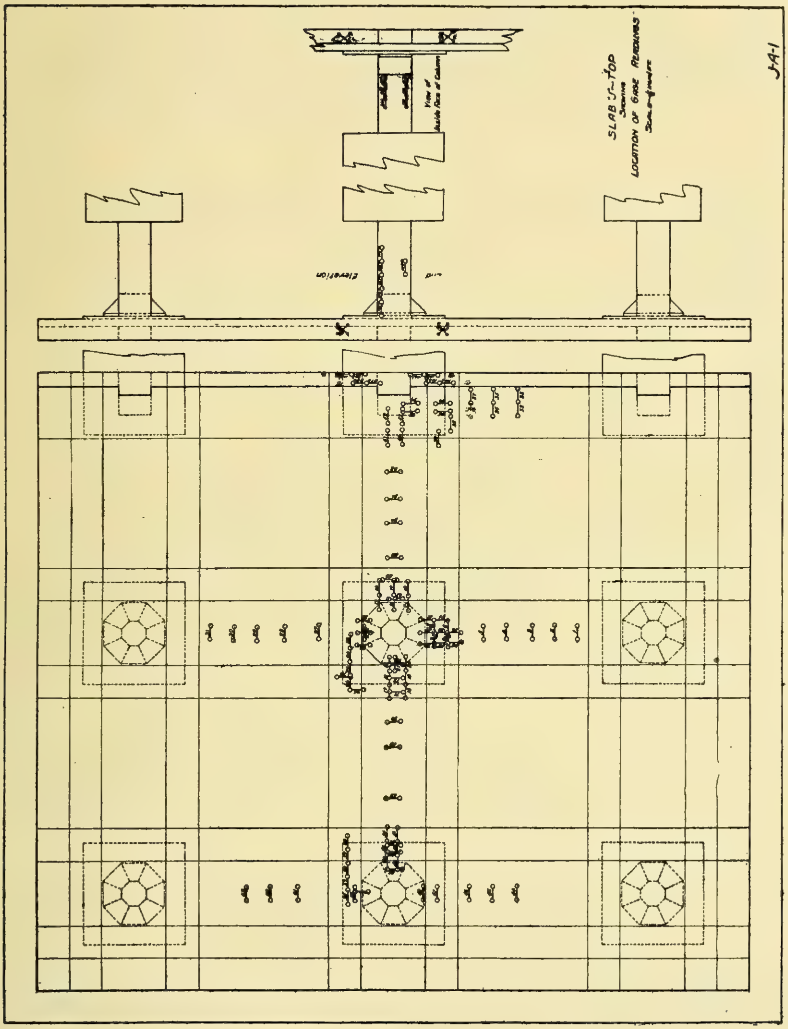


FIG. 8.—LOCATION OF GAGES, TOP, J SLAB.

These flat slab tests are shown in Table 1 and Figs. 2, 3 and 4, upon which are shown, for comparison, the results on a flat beam and on test panels of the flat-slab construction. The coefficients of Lord's tests are as worked out by him and communicated to the Joint Committee by letter (Nos. 3, 4, 5, 6 and 7).

TESTS OF SPECIAL TEST PANELS.

Recently the author has carefully tested certain flat-slab panels at Purdue University with the cooperation of the Corrugated Bar Company of Buffalo, N. Y., in which the objections cited in (3) were removed, and it is believed that the resisting moment as resident in the steel stresses was closely measured. The panels were fully loaded and the disturbing factors due to the resistance of the columns have been removed. These should show the relation of internal moment coefficients to the external moment coefficient, 0.09 in a flat slab, as computed from statical considerations only. The results are recorded in Figs. 2, 3 and 4 for positive,

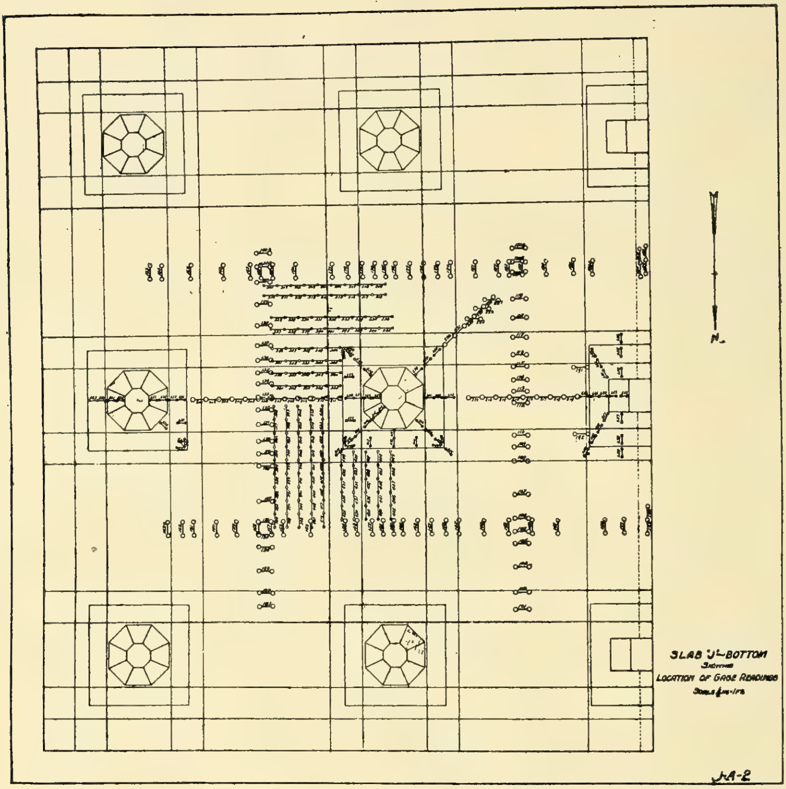


FIG. 9.—LOCATION OF GAGES, BOTTOM, J SLAB.

negative and total moment. On this same figure is shown the curve for a flat beam tested in the laboratory on a simple span in positive total moment.

Tests of several buildings may be expected to yield very different moment coefficients as determined by steel stresses at specific loadings of the floor surface. If the concrete is excellent the stresses and the coefficient will be low; if the concrete is poor, both stress and coefficient will be high. The thickness of the slab also influences the result.

A comparison of the values in Fig. 2 and Figs. 3 and 4 show that, so far as these quotations of tests establish a general truth, there is little difference between the behavior of a flat slab, a flat-slab panel, and a simple beam in the laboratory, after the concrete has broken down in tension at earlier loads. A portion of the difference is due to the nominal leverage of steel of 7/8 *d* taken for structures; whereas the actual measured leverage is used for the flat beam.

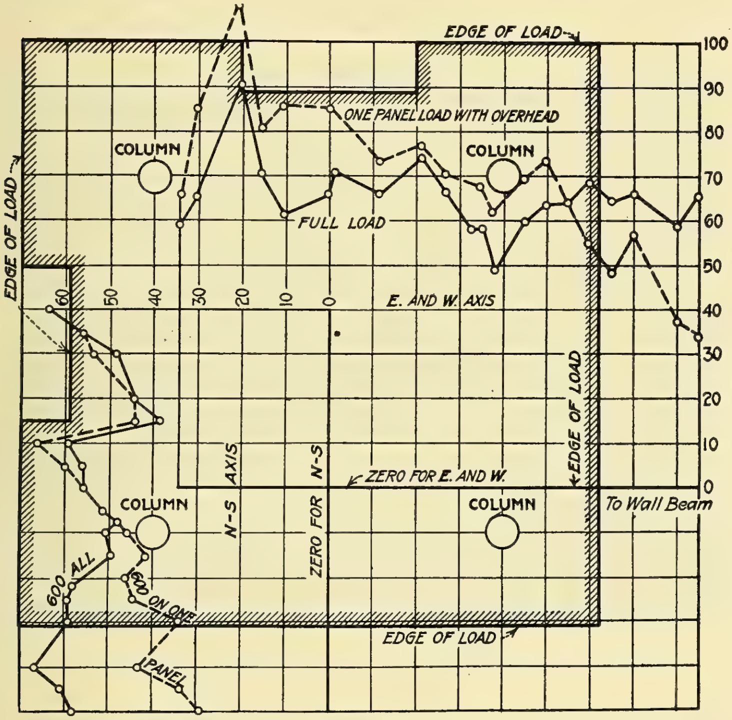


FIG. 10.—LOADING DIAGRAM—803-LB. LOAD, AND COMPARISON OF COMPLETE WITH ONE PANEL LOAD AT 600 LB.

Consequently, we may design these slabs upon the basis of a statical analysis, as recommended by the Joint Committee. And it follows that in the presence of poor concrete our expected stresses at assigned loads will be more nearly realized. We are concerned mainly with problems of distribution of moment throughout the slab, and efficiency of distribution of steel.

It must be admitted that the circumstances of pouring and hardening favor the production of a good quality of concrete in a flat slab, and permit higher working stresses, both in steel and concrete. One way of expressing this condition is in reduction of moment coefficient. The Joint Committee has reduced the moment from 0.09 Wl to 0.076 Wl total positive and negative, when the column head is 0.225 of the span, to allow for this favorable condition. Here W=load on entire panel and l=center distance of columns. This moment is to be provided around each of the right-angled axes.

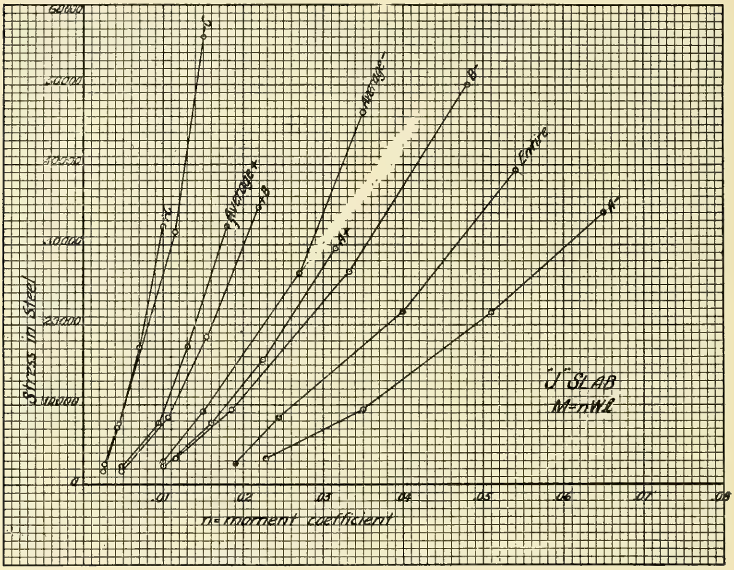


FIG. 11.—MOMENT COEFFICIENTS AND STRESSES, J SLAB.

But no matter how the portion of the resisting moment arising from tensile stresses is distributed between concrete in tension and steel in tension, the concrete in compression is but little affected by this division. And by fixing attention upon steel stresses alone and allowing low coefficients for these, we may perhaps forget that this process will bring high stresses on the concrete in compression near the column head.

Thickness of slab, quality of concrete, moment coefficients, all enter into the design of a satisfactory slab.

RESULTS OF TESTS OF "J" SLAB.

The above discussion of moment coefficients has mentioned certain tests an experimental flat-slab structure at Purdue University made in cooperation with the Corrugated Bar Co. of Buffalo, N. Y., represented at the tests by Mr. S. J. Hollister, Research Engineer. There were three slabs tested, in which there were variations in the amount and disposition of the steel. Concrete sizes remain the same in all.

It is expected that the entire investigation may be reported in all detail upon another occasion. At the present time the results reached upon the slab called "J" will be reported.

Slab "J" had steel disposed according to the arrangement of the Corr Plate floor, as designed by the Corrugated Bar Co., and the amount of steel was that necessary to meet the requirements of the moment coefficients recommended in the Joint Committee report.

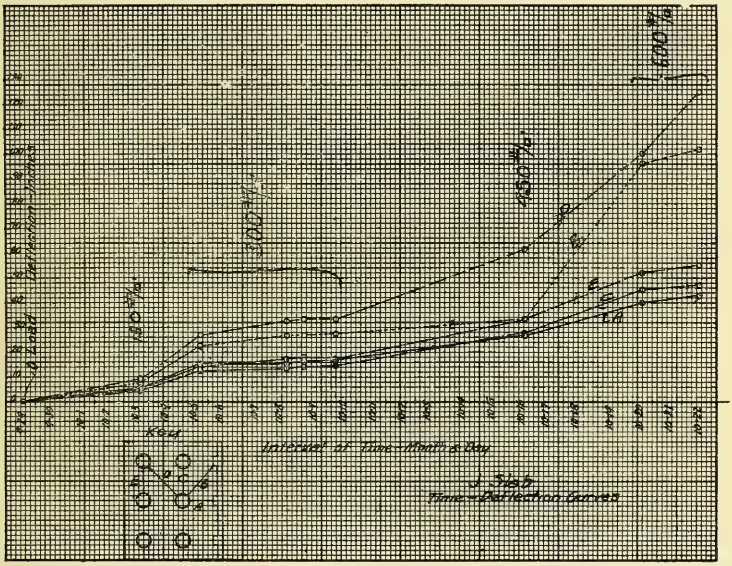


FIG. 12.—TIME-DEFLECTION CURVE.

The thickness of the slab was 5½ in. and the drop 7½, whereas the Joint Committee report would have required 5 8/10 in. and 8 7/10 in. respectively.

The dimensions of the slab are shown in Fig. 5 and the steel layout in Fig. 6.

For purposes of reference the bands of steel are referred to as:

Band A, 4 ft. wide over the column head.

Band B, 4 ft. wide, 2 ft. on each side of Band A.

Band C, 8 ft. wide in the center of the panel.

The steel amounts are as follows:

	POSITIVE STEEL, SQ. IN.	NEGATIVE STEEL, SQ. IE.
Band A	1.90	2.63
Band B	1.10	1.51
Band C	1.10	1.10
Total	4.10	5.24

The design load was 150 lb. per sq. ft.

The steel was hard grade steel with a yield point of 55,600 lb. per sq. in. The concrete had a compressional strength of 2300 lb. per sq. in.

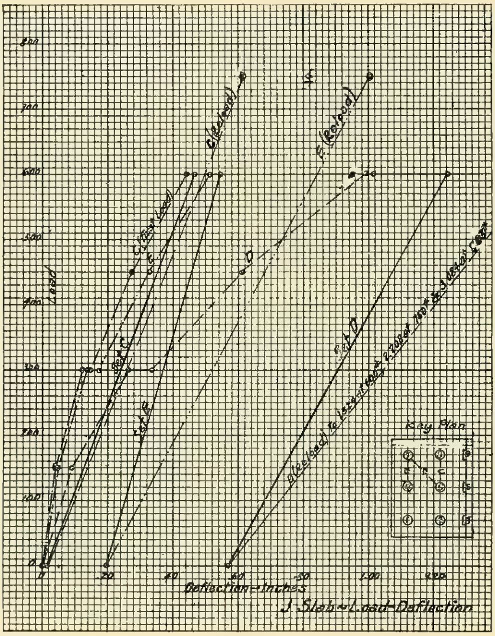


FIG. 13.—LOAD DEFLECTION CURVE.

at the age of 28 days and 2586 lb. per sq. in. at 95 days. The deformations of a test cylinder were used as indicative of stresses in the slab at the same deformations.

The slab was loaded at an age of 66 days.

The wall beam runs north and south.

Loading.—Brick was used as a loading material, and piled as shown in Fig. 7. Eight-inch aisles were left along panel centers. Tunnels were provided for observation of deformations as shown. The brick were piled along the sides of the tunnels to provide a moment that would come from the brick if piled in the aisles to the depth of the loading throughout the remainder of the slab.

The total load divided by the total area of the slab, outside the column caps, was, for the various increments (in lb. per sq. ft.):

Nominal	150	300	450	600	803
Actual	150	306	450	595	...

The loads of 150, 300, 450, 600 were over the entire slab. The load of 803 was over one panel and extended to the outer edge of Band B in the adjoining panels and over the overhang.

The overhang was 6 ft. wide and was greater than necessary to balance the moment over the columns, as shown by the readings of a level bubble placed over the top of column. The test of another slab showed that 4½ ft. was too small an overhang.

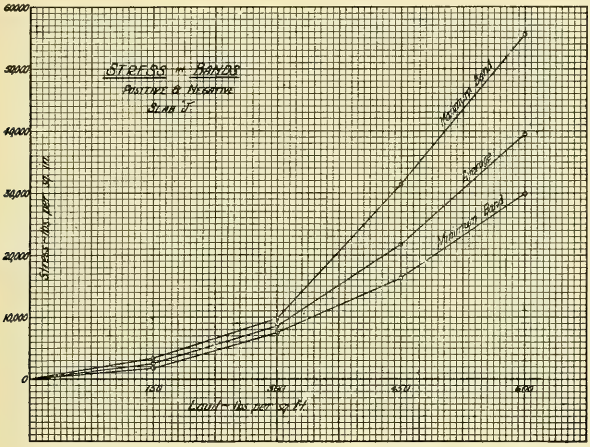


FIG. 14.—LOAD—BAND STRESS.

Gauges.—The location of gauges is shown in Figs. 8 and 9 to the number of 442.

Special Steel.—In addition to the customary steel, a series of small bars, ¼-in. square, were laid in the bottom of the S E and N E panels for one quadrant, as shown in Fig. 9. These were used to determine the variation of stress, as shown by the stress contours in Fig. 20.

The question arises if the extra compensating brick, piled alongside the housings in Slab J, did not overstrain the B bands and increase the moment on the margins; and also the question as to the effect of the special steel in disturbing the normal distribution of the stresses in the slab.

The fact that the margins increased their stress rapidly before bands A and B, does not indicate such a disturbance, for with two other similar slabs, without the extra brick or special steel, the margins acted similarly.

The sudden increase in negative stress of band C relative to the negative stress in bands A and B (at a stress of approximately 10,000 lb. per sq. in.) is characteristic of the design of these slabs.

Observations.—For steel stresses, the 10-in. Berry Extensometer was used, after calibration. For compression, a 6-in. and a 2-in. instrument were used. The value of one division of the dial of the 10-in. instruments was found to be 569 and 560 lb. per sq. in., respectively, on steel; and on concrete of 3,000,000 modulus, the value of one division of the 6-in. instrument was 93 lb. per sq. in. Temperatures were read of the air beneath and above the slab, and in the concrete. The extensometers were of Invar steel with a low coefficient of expansion. They were checked back on an Invar standard bar after a set of twenty readings. The standard bar had a coefficient of expansion of 1/7 that of ordinary steel. At this time also a reading was taken on a common steel bar embedded in a block of concrete, *i. e.*, the "calibration bar," to serve as a basis for correcting slab deformations for effects of change of temperature.

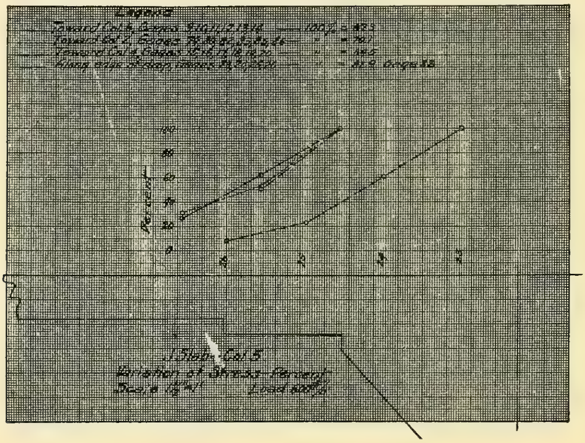


FIG. 15.—STRESS VARIATION, TOP STEEL, CENTER COLUMN.

That is, the slab deformations of the slab steel were due to, (1) load and (2) to change of temperature. The temperature deformations may be supposed to be nearly equal to the change of length of the "calibration bar." However observations made upon the changes of length of selected gauges in an adjoining unloaded slab due to change of temperature showed that a change of 10° F. would move the dial of the 10-in. instrument through two divisions. Since the observations of deformations under the various loads were made at different temperatures, and since the change of temperature occurring during a three-hour run must be corrected for, the observations were all reduced to a standard temperature of 60°. Thus the changes of gauge length and the stresses are freed from temperature effects, both in the slab steel and in the increments.

The matter is important as the change of temperature, likely to occur from early morning to noon, will cause deformations greater than those under the design load. Indeed, in the writer's experience, in case of a pair of tests on buildings that were to be compared, the temperature rose 10° between loads in one case and fell 27° in the other case.

TABLE 2.—STRESS AND MOMENT COEFFICIENTS IN J SLAB—INTERIOR PANEL —LIVE LOAD.

Band.		150 lb.		300 lb.		450 lb.		600 lb.	
		Stress.	n	Stress.	n	Stress.	n	Stress.	n
A	Positive N-S......	1,470		5,820		15,580		28,780	
	Positive E-W......	3,110		9,620		16,040		30,650	
	Aver...........	2,290	0.0100	7,720	0.0166	15,810	0.0225	29,715	0.0315
B	Positive N-S......	1,340		6,000		16,450		32,940	
	Positive E-W......	2,510		10,590		20,490		36,740	
	Aver...........	1,925	0.0048	8,295	0.0106	18,470	0.0154	34,840	0.0220
C	Positive N-S......	1,970		5,600		15,650		31,820	
	Positive E-W......	1,890		10,190		18,870		33,420	
	Aver...........	1,930	0.0024	7,895	0.0048	17,260	0.0070	32,620	0.0100
Positive, Whole Aver.......		2,048	0.0048	7,970	0.0095	17,180	0.0130	32,390	0.0184
A	Negative N-S.....	2,660		8,617		20,880		35,040	
	Negative E-W.....	3,590		10,495		21,775		33,020	
	Aver...........	3,125	0.0228	9,556	0.0349	21,328	0.0515	34,030	0.0655
B dr.	Negative N-S.....	2,490		8,430		24,410		45,740	
	Negative E-W.....	3,540		11,170		29,430		55,120	
	Aver...........	3,015		9,800		26,920		50,430	
B sl.	Negative N-S.....	2,260		8,120		24,230		42,750	
	Negative E-W.....	3,820		11,730		28,920		56,500	
	Aver...........	3,040		9,925		26,575		49,625	
B sl+dr	Negative..........	3,028	0.0114	9,863	0.0186	26,748	0.0332	50,028	0.0480
C	Negative N-S.....	2,170		7,170		33,890		56,290	
	Negative E-W.....	3,110		8,010		29,370		55,500	
	Aver...........	2,640	0.0027	7,590	0.0041	31,630	0.0114	55,895	0.0150
Negative, Whole Aver......		2,931	0.0100	9,000	0.0151	26,570	0.0270	46,650	0.0355
Entire + dr.		2,478	0.0148	8,480	0.0246	21,870	0.0400	39,500	0.0539

Plasticity.—Loads must be kept on long enough to allow the plastic action of concrete to run its main course. In the experience of the writer, the substantial effects are produced at the end of three days. In the case of this slab J, the interval of loading was 21 days from beginning to completion of the uniform loading.

Stresses.—The stresses are shown in Table 2.

Maximum Load.—The maximum uniform load was 595 lb. per sq. ft. over the whole slab. After this was entirely removed, the brick were piled to a load of 803 lb. per sq. ft. over one panel, the overhang, and into the adjoining panels to the further edges of b and B, as shown in Fig. 10.

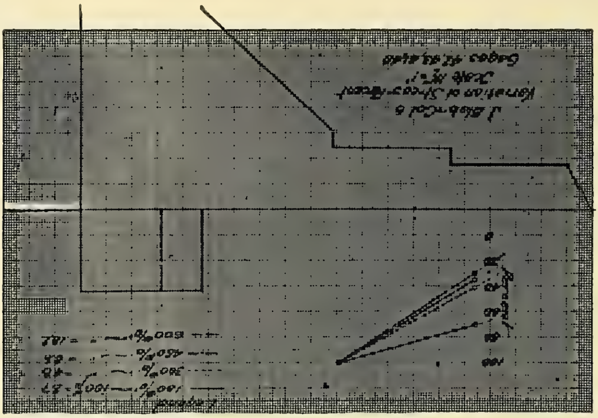


FIG. 16.—STRESS VARIATION, TOP STEEL, WALL COLUMN.

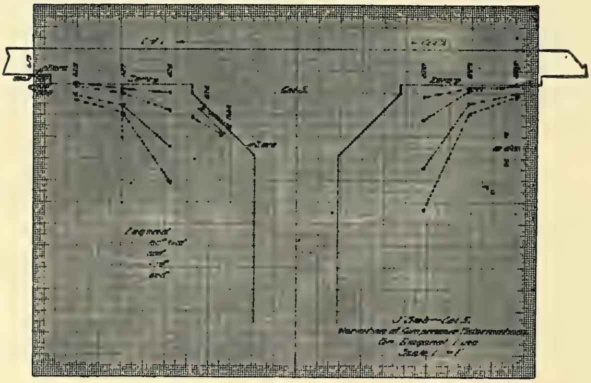


FIG. 17.—STRESS VARIATION, TOP STEEL, COMPRESSION, CENTER COLUMN.

When the load of 600 lb. was taken off the slab the large cracks were closed up, as they should, since the steel had not reached its yield point and there was no compression failure in the concrete.

The effect of the load of 803 lb. was to dish down the panel and crush the slab at the column heads. The failure of the slab at these column caps began with a feathering of the concrete on the drop at the edge of the column cap, followed by a diagonal failure in the drop around the head of the column. The punching shear at this stage was computed to be 198 lb. per sq. in.

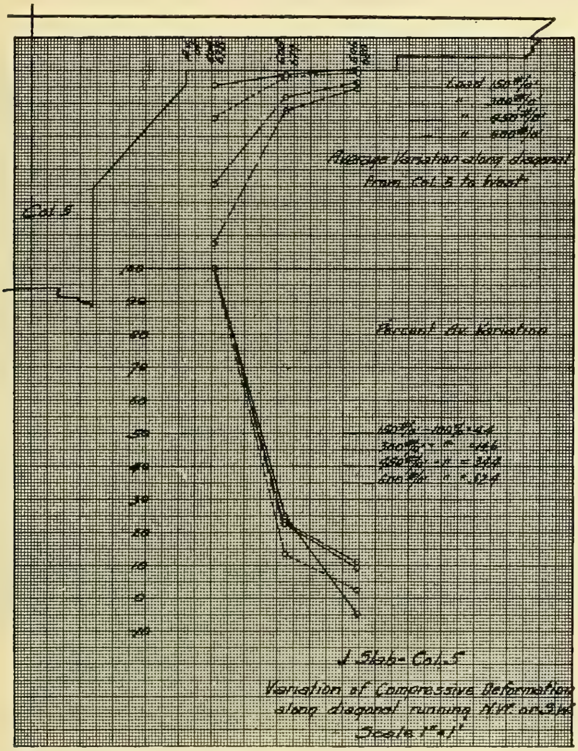


FIG. 18.—STRESS VARIATION, IN PER CENT.

At load of 600 lb. per sq. ft. the stresses were:

Average stress in band that was stressed to the greatest degree—56,000 lb. per sq. in.

Maximum compressive stress—1800 lb. per sq. in.

Ratio of yield point of steel to stress = 1.09.

Ratio of strength of concrete to maximum stress = 1.40.

With poorer concrete, the compressive strength of the concrete and the yield point of the steel would have been reached earlier.

The concrete might be called an average job, not so good as in the case of the buildings tested and referred to above.

Construction Joints.—The slab was poured in three sections with joints running E-W, or at right angles to the wall beam through the center of the panel. At a stress in the steel of 5800 lb. per sq. in., where the concrete was intact, the steel stress across a constructive joint running E-W was 75 per cent greater. At a stress of 32,000 lb. per sq. in. the stresses in both directions became substantially equal.

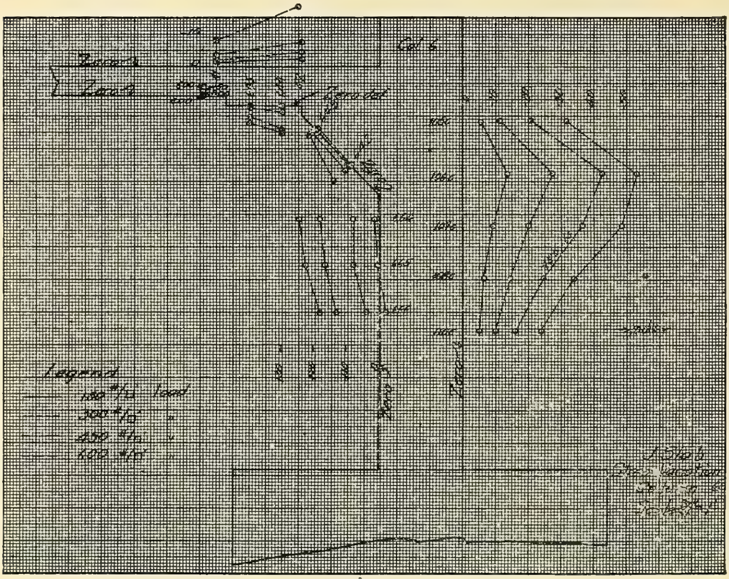


FIG. 19.—STRESS VARIATION, WALL COLUMN.

Distribution of Stress.—The ratio of negative to positive stress is as follows:

BAND.	LOAD 450.	LOAD 600.
A	1.35	1.14
B	1.45	1.43
C	1.86	1.72

Evidently the negative steel may well be increased, especially in the margins or mid-section.

The uniformity of stress is shown in the following, where the average = 100.

Range of minimum to maximum band stress:

	LOAD 450.	LOAD 600.
Positive	92 — 107	92 — 107
Negative	82 — 120	73 — 120

Deflections as a Fraction of the Direct Span (16 ft.).—

LOAD.	DIRECT SPAN.	DIAGONAL SPAN.
150	1/3800	1/1920
300	1/1370	1/735
450	1/580	1/315
600	1/375	1/194

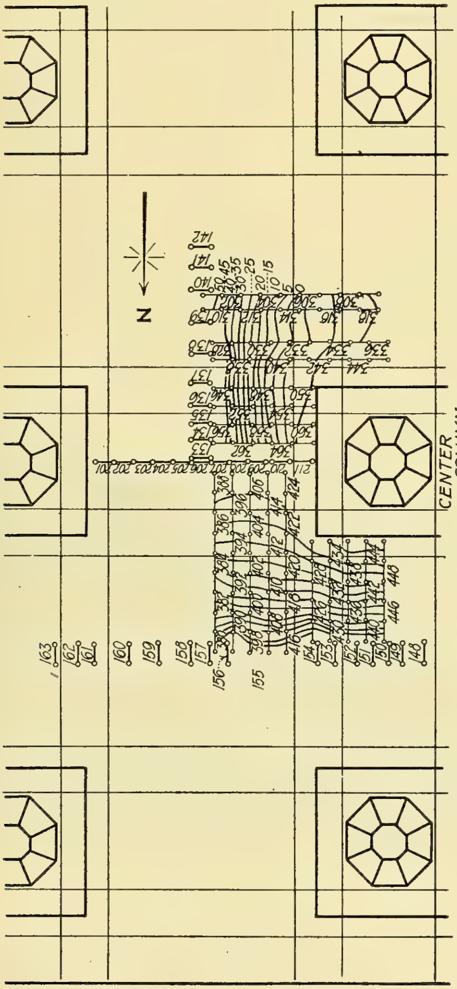


FIG. 20.—STRESS CONTOURS, SPECIAL STEEL.

By interpolation the deflection and stress at twice the live plus the dead load become:

Diagonal Deflection	1/525
Average Positive steel stress	12,570
Average Negative steel stress	17,270

These steel stresses are a function of the quality of the concrete.

The following ratios obtain where
LC = collapsing load.
LM = maximum uniform load = 595 lb. per sq. ft.
LD = design load (150) D = dead load (75).

$$\frac{LM}{LD} = 4 \qquad \frac{LM + \mathrm{D}}{LD + \mathrm{D}} = 3$$

$$\frac{LC}{LD} = 5.35 \qquad \frac{LC + D}{LD + D} = 3.44$$

$$\frac{\text{Yield Point of Steel}}{\text{Maximum Band Stress at } LD + D} = 11.2$$

$$\frac{\text{Ultimate Strength of Concrete}}{\text{Concrete Stress at } LD + D} = 6$$

Dead-load steel stresses were measured at selected locations by observations before and after removal of forms and found to be in the same proportion as the live load stresses.

Moment and Moment Coefficients.—The continued product of the area of the steel, the stress in the bar, and ⅞ of the depth of the steel from the surface of the slab yields the moment of resistance supplied by a given bar. The sum of all of these partial moments around an axis is the moment of resistance around that axis as supplied by steel stresses. Of course this moment of resistance is only part of the total moment, since the tensional stresses in the concrete are not accounted for.

Table 2 gives these coefficients for Slab J as follows:

	150	300	450	600
POSITIVE.				
A	.0100	.0166	.0225	.0315
B	.0048	.0106	.0154	.0220
C	.0024	.0048	.0070	.0100
All Pos.	.0048	.0095	.0130	.0184
NEGATIVE.				
A	.0228	.0349	.0515	.0655
B	.0114	.0186	.0332	.0480
C	.0027	.0041	.0114	.0150
All Neg.	.0100	.0151	.0274	.0355
Total	.0148	.0246	.0400	.0539

These coefficients are plotted in Fig. 11.

Distribution of Moment Coefficients.—The total coefficient is divided in per cent.

	SLAB J.				JOINT COMMITTEE ($M = .09\ Wl$).
POSITIVE.	450 LB.		600 LB.		
A	43.0		43.2		
B	29.7		29.6		
C	27.3		27.2		25.0 minimum
	100.0	32.3	100.0	34.0	37.8
NEGATIVE.					
A	47.5		45.2		
B	31.9		34.1		
C	20.6		20.7		20.0 minimum
	100.0	67.7	100.0	66.0	62.2

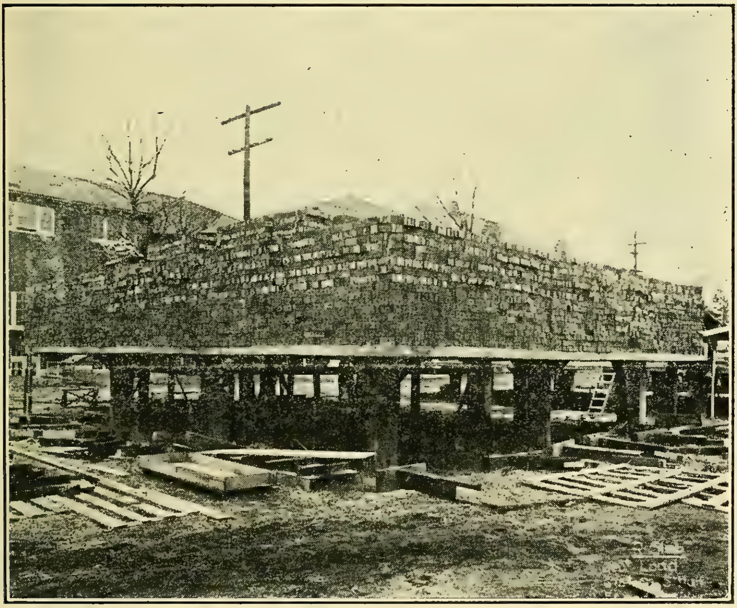


FIG. 21.—VIEW OF R SLAB WITH 600-LB. LOAD.

Averaging the tests of Slab J and a similar slab with less steel, the moment was divided nearly into 65 per cent negative and 35 per cent positive, instead of the values 62.2 and 37.8, fixed by the Joint Committee. These tests also indicate the necessity of the minimum steel in the mid-section (C negative) and in the inner-section (C positive) required by the Joint Committee.

Load Carried by Lintel Beam.—The steel stresses at the center and at the support of the lintel beam allow computation of the total moment of resistance of this beam. This maybe equated to $\frac{1}{8}\ Wl$, and the value of W, the load causing the flexure of the beam, may be computed. In Slab J, at 600 lb. per sq. ft. loaded on the panel, the indicated load on the lintel beam was 0.121 of the total panel load. Another slab, with more highly developed steel stresses, yielded a fraction of 0.16.

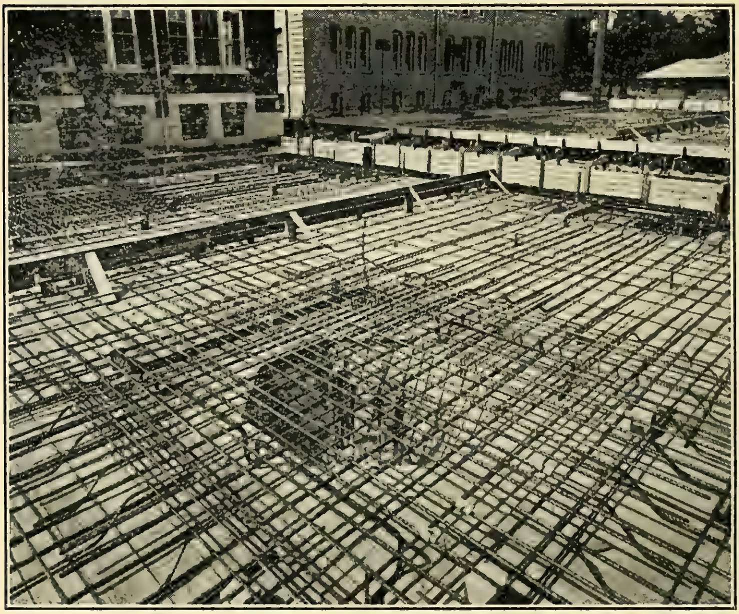


FIG. 22.—REINFORCEMENT OF J SLAB.

The moment in the wall column was in a similar slab $0.02\ Wl$ where $W =$ load on panel and $f =$ panel length.

Comparison of Wall and Interior Panel.—The ratio of positive stress in wall and positive stress in interior panel was:

	450.	600.
A	0.75	0.73
B	0.91	0.82
C	1.01	0.93
Average	0.89	0.83

The 20 per cent additional steel in the wall panel positive sections is evidently sufficient.

The stresses in the negative steel running into the wall beam, 2680 lb. per sq. in., and into the head of the wall column 10,230, are, at 600 lb. load, low compared to the negative stresses in the interior panel. Of course, this is only an expression of the yielding of the wall beam and wall column.

Measurements on the web of the lintel beam gave consistent indications of torsion.

The distribution of compressive stresses over the drop panel shows approximately that the concrete compressive stress at the edge of the drop is only about 25 per cent of the stress at the edge of the head of the column. In considering the allowable unit stress computed as an average over the drop panel, the maximum that fixes the strength of the slab must be considered.

Various diagrams are reproduced showing distribution of steel and concrete stresses and photographs of the slabs.

DISCUSSION.

Mr. Martin. Mr. E. S. Martin.—I notice that the first paragraph of this paper contains a statement that is rather an old adage; that whatever there may be to the theoretical analysis of the homogeneous flatplate, of steel for instance, it may not be applied with equal force, or truth in fact, to one of reinforced concrete. It was not many years ago that the question of the theoretical analyses was first discussed, and since then it has been my position that theories do not really have more than academic value and that a much more reliable basis for studying the flat-slab can be obtained from numerous tests. It does not seem that there should be any distinction between the reinforced-concrete flat-slab and the homogeneous flat-slab. In general, the reinforced slab is designed on the assumption that the reinforcement supplies the tensile strength and the concrete the compressive strength. When that premise is made, a corollary is that there is a direct connection, a bond stress between the steel and the concrete in order to make the member, beam or slab as the case may be, act as a unit, and in doing its work the reinforcement must cause the surrounding concrete to take shearing stress.

The peculiar stress of the flat-slab theory is nothing more than the inner action which takes place between the different stressed portions of the slab, that is between the different parts as regards each other in a horizontal plane. This inner action arises out of the shear between the different parts, so long as the flat-slab is not broken up, but acts as a unit. It does not occur to me that there is any reason for saying that this action does not take place in a flat-slab of concrete as well as in a homogeneous one. The action may be nil or very small but it cannot be passed over without study and it does not take place in one any more than in another.

This series of tests does not seem to show a very great difference in the proportion of the moment sustained by the steel and that by the tensile strength of the concrete for the variations in percentage of steel reinforcement. On the other hand, other series of tests have seemed to show quite a marked increase in the proportion of moment sustained by the steel where the percentage is high as compared to where it is low.

Another point that has been brought to my attention in the study of flat-slab tests is the very extreme compressive stresses in the concrete immediately adjacent to the capital. All the early flat-slab buildings were designed without what is called the depressed panel, and in such buildings the compressive stress in the concrete—at least in the early buildings where the thickness of the slab was uniform—ran up very high, to two or three times the allowed compression. It occurs to me that hardly sufficient attention or emphasis has been placed on this by the Joint Committee and also by the various building department regulations. They have not set forth a very accurate criterion as to what the thickness and spread of this depressed panel should be.

MR. D. A. ABRAMS.—The relation between the moment of the slab and the calculated stress in the steel as brought out in this paper recall the test made some years ago at the Southern Illinois Prison and reported in the 1914 Proceedings of the American Society for Testing Materials. This test was on a full-size slab and girder concrete bridge. Careful measurements were made during loading and it was found that about 25 per cent of the total calculated moment in the slab could not be accounted for by the stresses in the steel. The discrepancy was laid to the concrete carrying tensile stress, although the bridge was loaded to thirteen times the designed load and the floor to nine times the designed load. **Mr. Abrams.**

MR. W. K. HATT.—In these particular tests, the interest lies in the distribution rather than in the total moment. **Mr. Hatt.**

Tests made more recently have brought out clearly the advantage of high carbon steel. In one test with high carbon reinforcement, wide cracks appeared at a load of 600 lb. per sq. ft. I think if the steel had been soft, the cracks would have kept open after the load was removed, but in this case after the load was taken off there was life enough in the steel to draw the slab back into position and close up the cracks.

TEST OF AN EIGHT-YEAR-OLD FLAT-SLAB FLOOR OF THE WESTERN NEWSPAPER UNION BUILDING.

By Arthur N. Talbot* and Harrison F. Gonnerman*.

The test of a flat-slab floor of the Western Newspaper Union Building at Chicago was made in August and September, 1917. As the building was to be torn down to clear the site for the new Union Passenger Station a very unusual opportunity was afforded of testing a floor with a load well beyond the usual test load. The test of the floor was unique also in that it was made on a structure built eight years before the time of the test. The loading was carried to a point which gave markedly higher stresses than had been obtained in previous building tests.

The Western Newspaper Union Building was an eight-story reinforced-concrete structure located at Clinton and Adams Streets, Chicago; it was erected in the spring of 1909.

Two types of floor construction were used in the building. The first five floors were slab and girder type; the sixth, seventh, and eighth floors were Turner "Mushroom" flat-slab type (four-way reinforcement). The floors of the building were divided into panels 17 ft. 5½ in. by 19 ft. 4½ in. The test was made on the sixth floor. This floor was designed for a live-load of 250 lb. per sq. ft. The thickness was nominally 8½ in., but the measured thickness at points within the test area ranged from 7.5 to 9.8 in. The interior columns were octagonal in form, 24 in. in short diameter below the floor tested and 21 in. in short diameter above it. The inside diameters of the hooping of the columns on the fifth and sixth floors were given on the plans as 21 in. and 18 in., respectively. The column capitals were pyramidal; the short diameter at the top of the capital was 54 in.

The building plans called for 15⅝-in. round bars in each of the four bands of reinforcement and indicated that over most of the columns in the test area there were laps in certain bands. After the test was made, the floor was broken into and the location of all laps and the position of the reinforcing bars with respect to the surfaces of the slab were found. Fig. 1 shows the arrangement of the reinforcement including the position of the laps. In several places the arrangement of reinforcement differs from that given in the building plans. In three places in rectangular bands the reinforcement for positive moment was double that given on the plans. The number of bands lapped at the columns was greater than was indicated; at column 15 three bands were lapped. In each column head two circumferential ring rods were used, and the eight column bars were bent out into the floor.

The concrete in the slab was 1:2:4 mix; in the columns 1:1:2 mix was used. The coarse aggregate was gravel. Pieces of the concrete were cut out from the floor and sawed into test prisms approximately 5 in. square and 16 in. long. The compressive strength of these test prisms ranged from 3190 lb. per sq. in. in panel B to 5460 lb. per sq. in. in panel A, and the initial modulus of elasticity from 3,500,000 to 5,100,000 lb. per sq. in.

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* University of Illinois, Urbana, Ill.

Tension tests of steel coupons cut from the reinforcing bars in the slab of the loaded area gave an average yield point by drop of beam of 63,600 lb. per sq. in. and an average ultimate strength of 101,300 lb. per sq. in.

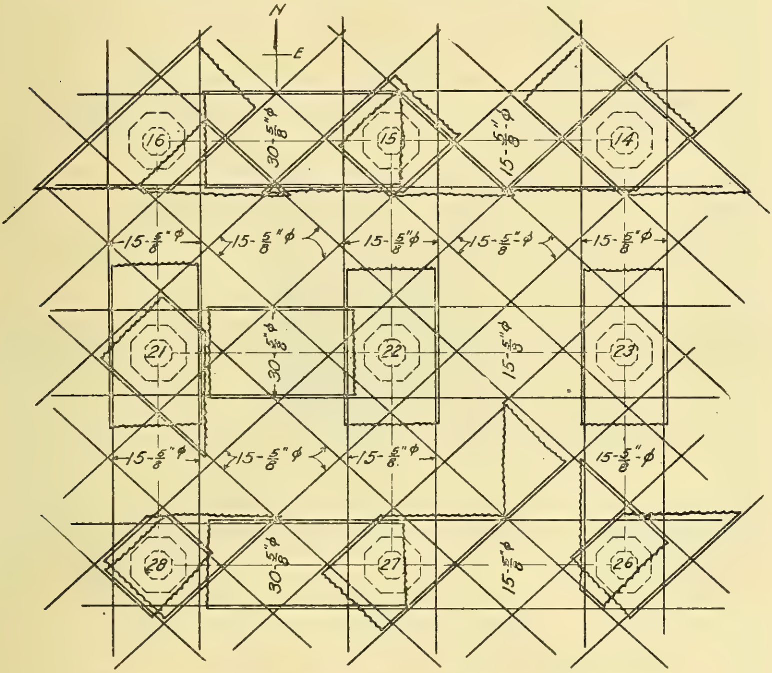

FIG. 1.—ARRANGEMENT OF REINFORCEMENT IN TEST AREA.

How the Test was Made.

The method of testing was similar to that used in previous building tests, as described in Bulletin No. 64 of the University of Illinois Engineering Experiment Station, "Tests of Reinforced-Concrete Buildings Under Load." Gage lines were prepared in advance of the test,—103 on the reinforcing bars and 75 on the concrete. To insure reliability of initial readings, three sets of strain-gage readings (more on many of the gage lines) were taken before the load was applied. Fig. 2 gives the location of the four panels on which the load was placed. Strain-gage readings were taken on the reinforcing bars and on the concrete after each increment of load was placed. The deflection of the slab was measured at 20 points.

The loading material was pig iron. The pig iron was hauled from the freight yards to the building in motor trucks. The net weight of each truck load of iron was obtained by weighing on a certified scale before it was hauled to the building. A record was kept of the number of truck loads placed on each panel and the total weight on each panel was obtained from the truck weights. The load on each panel was divided into quarters by means of aisles 6 to 8 in. wide extending at right angles to each other along the center lines of the panels, and along the boundaries between panels. The space occupied by the aisles and by the boxes built on the floor to afford access to the gage lines amounted to 17 per cent of the panel areas.

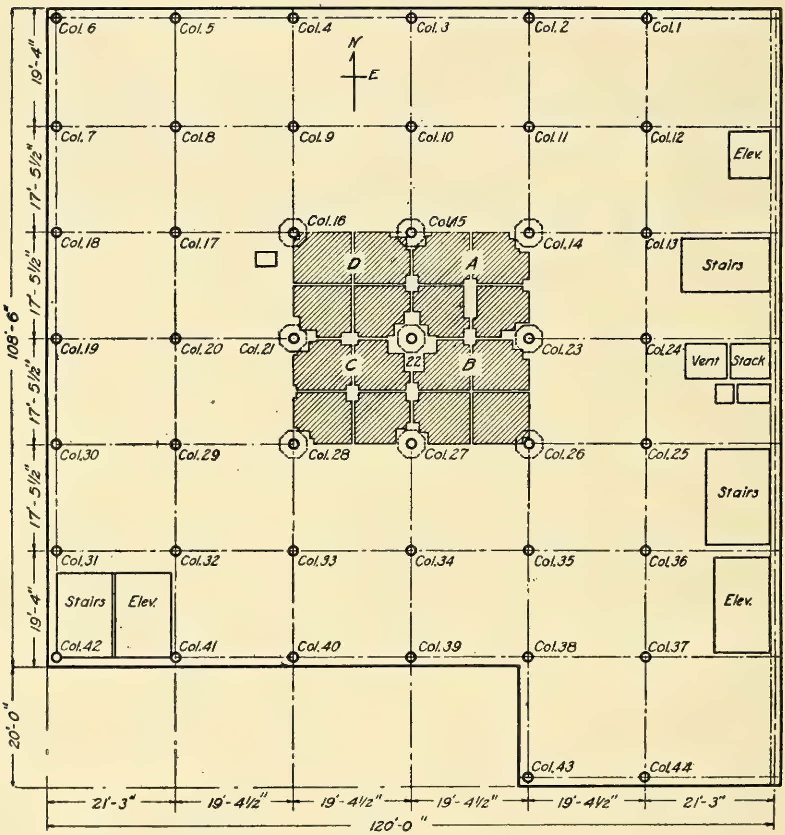

FIG. 2.—LOCATION OF TEST PANELS.

The loads at which strain-gage readings were taken were 234, 425, 637, 855, and 913 lb. per sq. ft. of panel area.

Readings were also taken three days after the maximum load had been placed on the floor in order to get information on the time effect of the load on deformations and deflections. After the removal of the load, readings were taken to find the amount of recovery in the floor.

DEFLECTION OF SLAB.

The deflections of the slab at eight points are plotted in Fig. 3. The recovery of the deflection one day after the load was removed is shown by the points plotted at the bottom of the diagram. It is seen that the greatest deflection under the load of 913 lb. per sq. ft. was 1.12 in. This deflection was observed at the center of panel B. The arrangement of the bars in the middle of this panel was different from that in the other panels and the concrete specimens taken from this panel were less stiff than those from the other panels; these differences may account for the greater deflection observed in this panel.

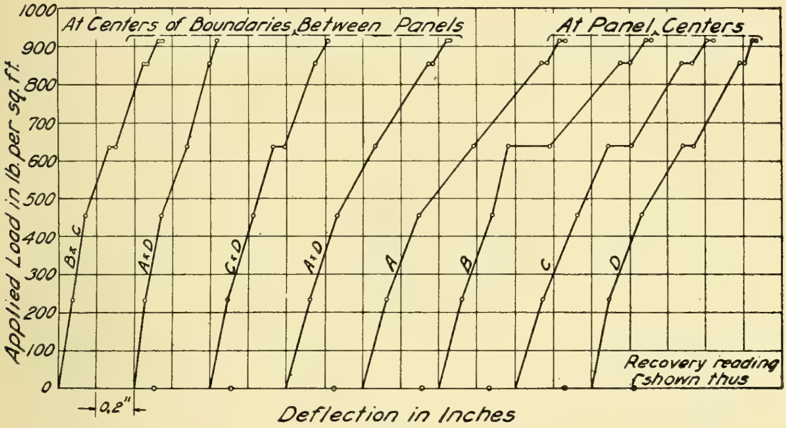

FIG. 3.—DEFLECTION OF SLAB.

CRACKS IN SLAB.

Fig. 4 gives the location of the more important cracks on the upper side of the slab which either opened up or formed with the application of the load. These cracks were all open cracks—much more marked than hair cracks. They ranged in width from 0.02 to 0.06 in. They show the regions of high tensile stress. The cracks which opened up over the edge of the capitals of the columns bordering the loaded area were apparently as large as those which formed over the edge of the capital of the central column. Many of the cracks shown in Fig. 4 were old cracks which were in the slab before loading was begun and opened as load was applied.

Fig. 5 shows the location of the more important cracks on the lower side of the slab for panels A and C, the panels in which the principal gage lines were placed. Cracks similar to those shown for panels A and C were also noted in panels B and D. In panel D the cracks were neither very wide nor very numerous even at the maximum load. In panel B the cracks were more numerous and were larger than those in any of the other panels.

Upon the removal of the load the cracks closed, leaving the surfaces of the slab with the appearance which they had before the load was applied.

STRESSES IN REINFORCING BARS.

The results of the strain-gage measurements have been plotted and studied. The diagrams show a good degree of concordance for measurements made in a building test and indicate that the results can be accepted with confidence. A few typical load-strain diagrams are shown in Fig. 6.

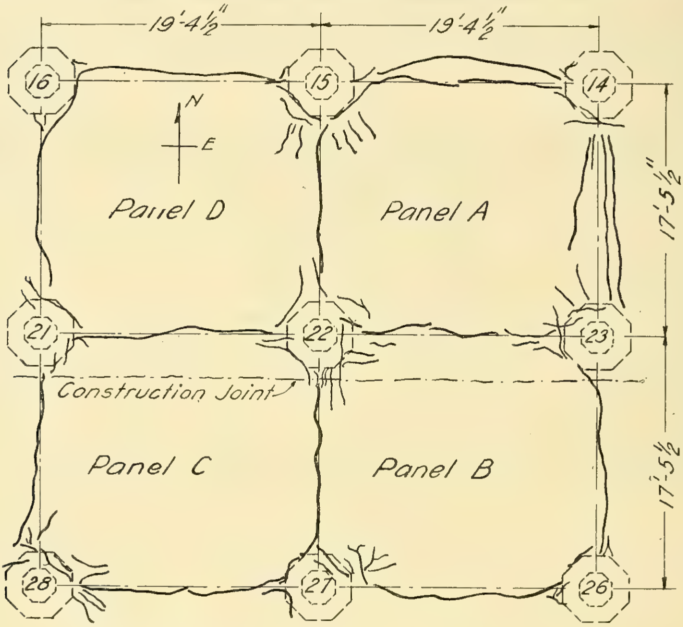

FIG. 4.—LOCATION OF CRACKS IN UPPER SIDE OF SLAB.

The stresses observed in the reinforcing bars on both the upper and the lower side of the slab at the maximum load of 913 lb. per sq. ft. are given in Fig. 7. For reinforcing bars on the upper side of the slab the greatest stresses were observed at the gage lines located on diagonal bars at the edge of the column capitals. At column No. 22 (the central column of the loaded area) the stresses in the diagonal bars over the edge of the column capital ranged from 49,200 to 57,300 lb. per sq. in. The stresses in diagonal bars at column 22 at gage lines located some distance from the column capital ranged from 15,000 to 34,500 lb. per sq. in. The stresses in the diagonal bars at the columns bordering the loaded area were nearly as great as those observed at corresponding points at the central column.

The stresses in the east and west rectangular band at column 22 averaged about 41,400 lb. per sq. in. The observed stresses in the north and south rectangular band at column 22 ranged from 23,700 lb. per sq. in. at the edge of the column capital to 40,500 lb. per sq. in. at gage lines located some distance from the capital on either side of the column. The bars in this band were lapped at column 22, the bars ending about 80 in. north and south of the column center. At gage lines on the rectangular bands located some distance from the edge of the capital of column 22 the stresses were greater than those at the edge of the capital. At sections of negative moment stresses as great as 6000 lb. per sq. in. were observed in bars near the edge of the rectangular bands outside the loaded area. It is evident that portions of the floor outside the loaded area contributed measurably to the resistance developed in the slab.

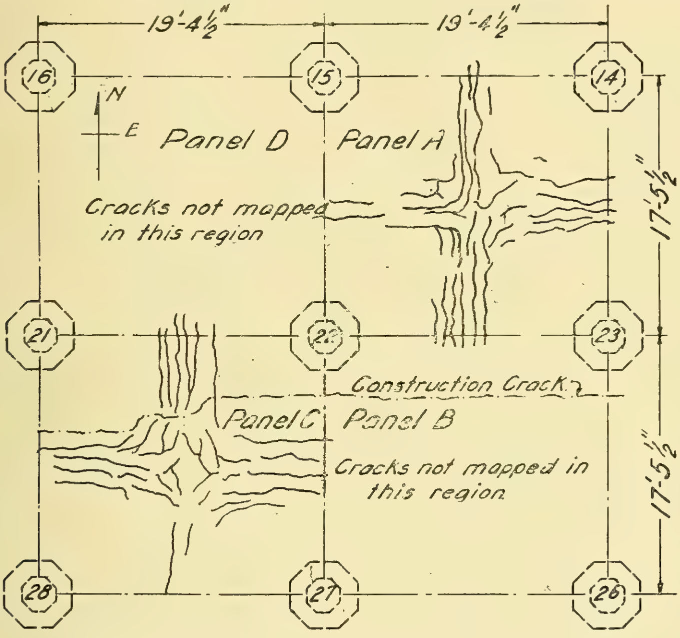

FIG. 5.—LOCATION OF CRACKS IN UNDER SIDE OF SLAB.

On the lower side of the slab the greatest stresses were found in the bars of the rectangular bands. At the maximum load stresses from 24,300 to 30,000 lb. per sq. in. were observed in rectangular bands within the loaded area. In the rectangular band within the loaded area which had 30 bars, stresses from 18,000 to 21,000 lb. per sq. in. were observed; the stresses in the rectangular bands at the edge of the loaded area varied from 5100 to 22,500 lb. per sq. in.; at one gage line outside of the loaded area and near

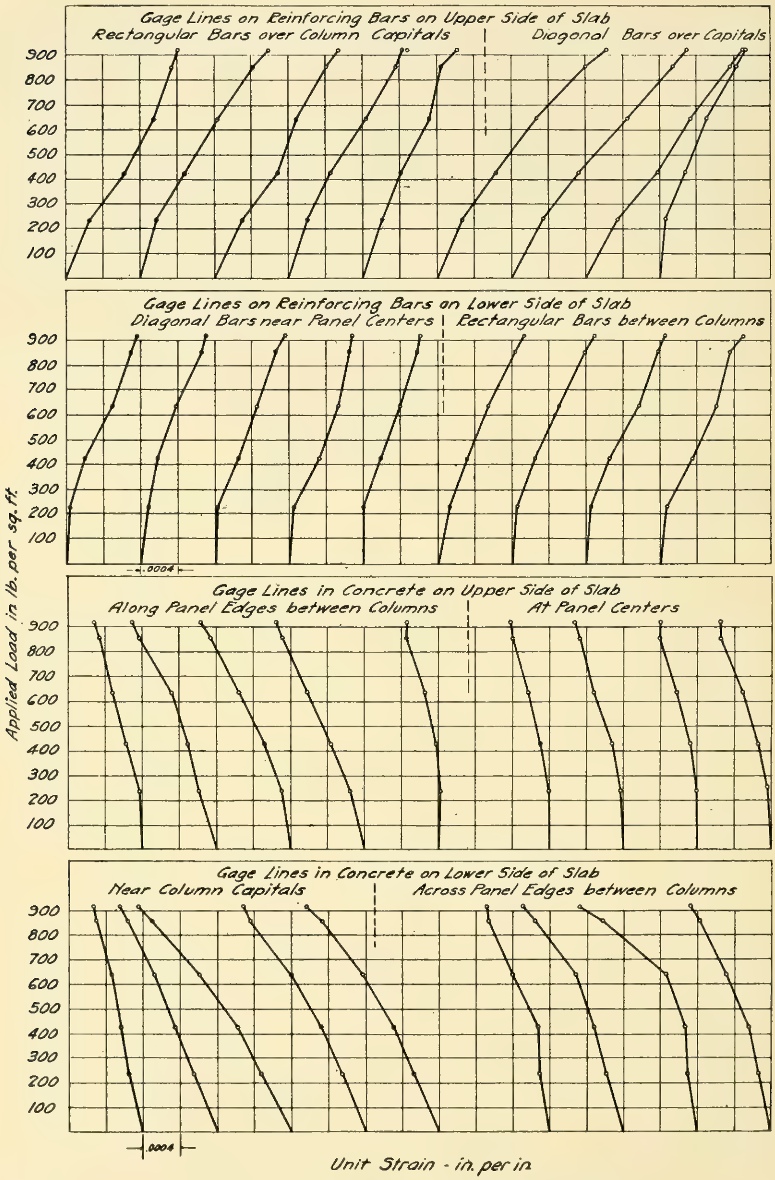

FIG. 6—TYPICAL LOAD-STRAIN DIAGRAMS.

the edge of the band a stress of 15,600 lb. per sq. in. was found. The stresses in the diagonal bands at the center of the panels were less than those in the rectangular bands between columns. The greatest stresses in the diagonal bars ranged from 17,700 to 24,000 lb. per sq. in. The stress in diagonal bars at sections away from the panel centers near the edges of the diagonal bands ranged from 6,300 to 18,300 lb. per sq. in.

STRAINS IN CONCRETE.

On the upper surface of the slab the greatest compressive strains or deformations were observed at gage lines located along the inner panel edges half way between columns. Strains from 0.00089 to 0.00097 in. per in. were observed at these gage lines. Assuming a modulus of elasticity of concrete of 4,000,000 lb. per sq. in. and a straight-line stress-strain relation the concrete stress corresponding to these strains would be 3560 to 3880 lb. per sq. in. Strains as great as 0.00054 in. per in. (corresponding concrete stress on assumption given above 2160 lb. per sq. in.) were observed at gage lines located at the panel centers.

The greatest strains in concrete on the lower surface of the slab were observed close to the edge of the capital of column 22. The strains at this column at the diagonal gage lines ranged from 0.0012 to 0.0016 in. per in. at the maximum load. These deformations are as great as the deformations which were found at failure when the concrete prisms cut from the floor were tested in compression; they are as great as the deformations usually found at the ultimate load in compression tests of concrete. In one of the panels spalling and chipping of the concrete surface was plainly visible near the edge of the column capital. At rectangular gage lines near the capital of column 22 a strain in the concrete of 0.0014 in. per in. was observed. These high strains indicate that the concrete near the capital of column 22 was highly stressed and that at certain gage lines it was stressed to its ultimate strength, though the action of the surrounding concrete prevented failure. Near the edges of the capitals of columns bordering the loaded area strains from 0.00054 to 0.0011 in. per in. were observed at diagonal gage lines, and strains from 0.00044 to 0.00081 in. per in. at rectangular gage lines. With a modulus of elasticity of concrete of 4,000,000 lb. per sq. in. the stresses corresponding to these deformations would range from 1760 to 4400 lb. per sq. in.

Strains as high as 0.0012 in. per in. were observed at gage lines crossing the inner panel edges away from the column capitals, even though there was no tension reinforcement in the upper side of the slab in this region. These high deformations indicate that the concrete was highly stressed along the section of negative moment away from the column capital. It should be added that there were deformations of some amount in regions of negative moment outside the loaded area.

RESISTING MOMENT ACCOUNTED FOR BY STRESSES IN REINFORCEMENT.

It may be of interest to compute the magnitude of the resisting moment accounted for by the stresses in the reinforcing bars and to compare it with the value of the bending moment due to the external forces. For this purpose a section of negative moment was taken in a north and south direction along the interior edge of the panels and skirting the peripheries of three column capitals, and a section of positive moment across the panels midway between columns, both sections extending to points outside the loaded area. Similarly, sections of negative moment and positive moment were taken in an east and west direction. The average of the stresses at the principal critical gage lines was used. For the bands at the edges of the loaded area the stresses were considered to vary over the band from gage line to gage line. Lapped bars were considered as contributing to the resisting moment wherever the bars extended beyond the section a sufficient distance to give adequate anchorage with respect to the magnitude of the stress developed in the bar, but many of the laps were thought not to be effective. Some judgment had to be used in regard to the availability of lapped bars and the distribution of

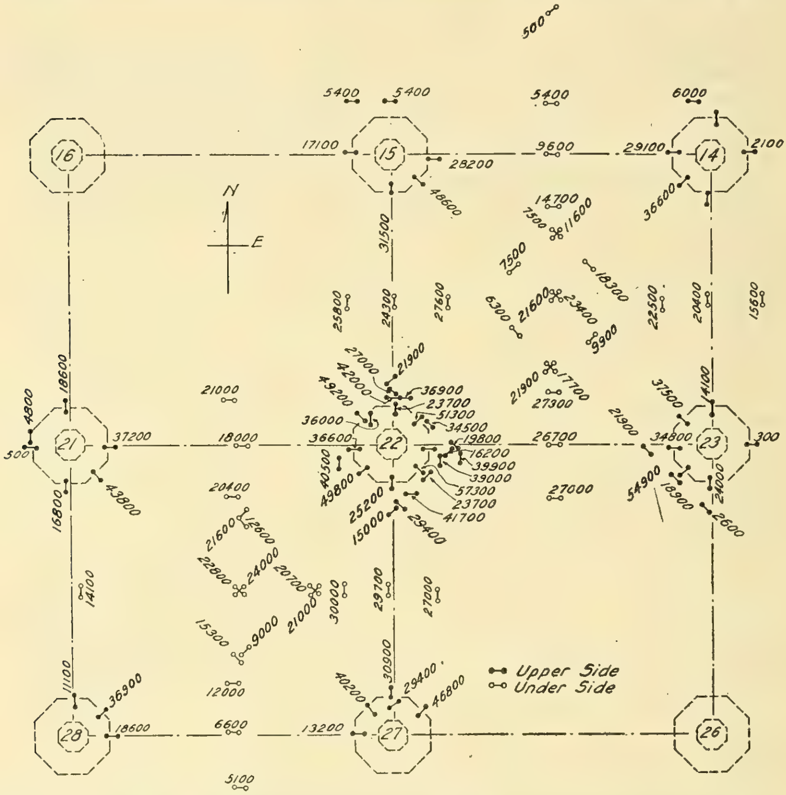

FIG. 7.—STRESSES IN REINFORCING BARS IN LB. PER SQ. IN. AT LOAD OF 913 LB. PER SQ. FT.

stress, but it is believed that the assumptions made give results well within the actual resistance developed in the slab. For the diagonal bars the component of the stress was taken at right angles to the direction of the panel edge. The measured position of the bars with reference to the face of the slab and the measured thicknesses of the slab were used. The calculation of the resisting moments due to the stresses in the reinforcing bars was then made in the usual way.

The resisting moments thus determined were compared with the bending moment found by multiplying one-half the load on two panels by the distance from the center of gravity of the load on the two half panels to the position of the resultant of the shears at the column capitals on which the two half panels rest, as given by the formula

$$M = \frac{2}{8} Wl \left(1 - \frac{2}{3}\frac{c}{l}\right)^2$$

where W is the load on one panel, l is the panel length in the direction at right angles to the section considered and c is the diameter of the column capital. Two-thirds of this analytical bending moment was considered as producing negative moment and one-third as producing positive moment.

On the basis thus outlined, the observed stresses in the reinforcing bars accounted for about 88 per cent of the analytical negative moment in the case of the north and south section and about 92 per cent in that of the east and west section. Similarly, about 68 per cent of the analytical positive moment was accounted for at the north and south section and about 72 per cent at the east and west section. Averaging the percentages for the two directions, 90 per cent of the negative moment and 70 per cent of the positive moment were accounted for.

The part played by the tensile stresses in the concrete is unknown and uncertain. It evidently contributes somewhat to the resistance of the slab, especially in parts of the sections of positive moment where the stresses in the bars are relatively small, as at the borders of the panels. At the cracks the intensity of the stress in the bars may be expected to be greater than the average stress over the gage length, the stress which is reported. Measurements made in beam tests in various laboratories generally show that up to loads near the ultimate load the measured stress is less than that necessary to account for the full bending moment due to the applied load, the deficiency decreasing as the stress in the reinforcing bars is increased; at stresses of 20,000 lb. per sq. in. the deficiency may be considerable. The effect is particularly noticeable in concrete of high quality.

On the whole the analytical values of the moments are closely approached, as closely as may be expected in tests of this kind. The negative moment of course is most fully accounted for. The positive moment is not wholly accounted for; but as the stresses at the sections of positive moment are relatively low the effect of the tensile resistance of the concrete may be considerable, especially in the part of the slab outside of the loaded area, and it must not be overlooked that the stresses in the bars at cracks will be larger than the average stress over the gage length.

The foregoing comparisons have been made on the basis of the full analytical value of the bending moment and by considering one-third of it as positive moment and two-thirds as negative moment. The Joint Committee on Concrete and Reinforced Concrete recommended a value of the moment which is about 85 per cent of the analytical value and a distribution as three-eighths positive moment and five-eighths negative moment. It may be of interest to note that the sum of the positive and negative moments accounted for by the measured stresses in the reinforcing bars has almost the same value as the sum of the moments recommended by the committee. The negative moments so accounted for are about 113 per cent of the moment recommended by the Joint Committee, and the positive moments are about 73 per cent of the committee's value.

In making a comparison with methods used in design it should be borne in mind that the principal observed maximum stresses were from 15 to 25 per cent greater than the average of the observed stresses which were used in computing the resisting moments accounted for by the stresses in the bars; in designing, a uniform stress over the section is assumed.

TIME EFFECT OF LOAD AND RECOVERY UPON REMOVAL OF LOAD.

The effect of time on the strains in the steel and concrete was very small; the reading for a few of the gage lines increased slightly after 12 hours, but hardly more than may be considered to be within the error of observation. This was true for both steel and concrete even in the most highly stressed places at the maximum load after a period of 66 hours had elapsed.

The deflection on readings were affected but little through a 12-hr. period until a load of 637 lb. per sq. ft. was reached, when a marked change in deflection occurred at the centers of panels B, C, and D in 12 hours' time. At the maximum load the increase in deflection through a 66-hr. period was small.

Four days were consumed in removing the load. Four hours after the last of the load was removed readings were taken on the gage lines and deflection points, and twenty hours later another set was taken. The cracks in both upper and lower surfaces of the slab had closed so that they were not easily traced. The recovery in deflection at the centers of the panels was about 75 per cent; at other points it was generally greater. The recovery in strains in steel and concrete was also large. Even where high stresses and open cracks had been observed the residual strain in the steel was not more than that to be expected through the prevention of interlocking of particles. In regions of high compressive stress in the concrete the recovery at the principal gage lines was considerable.

GENERAL COMMENTS.

The concrete in the slab gave evidence of high quality throughout the test, though there were differences apparent in the different panels.

With reference to the design of the slab, it may be noted that the slab was strongly reinforced, though the reinforcing bars were not distributed to the best advantage and the laps were not placed so as to be fully effective.

Taking the total reinforcement found over the loaded area and immediately outside, and including such lapped bars as had sufficient anchorage beyond critical sections to be effective (and there were many lapped bars which were not counted as effective), the amount of reinforcement for negative moment, considering the thinness of the slab, was on the average as much as that required for the negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete. The amount of reinforcement available for positive moment was on the average more than 50 per cent greater than that required for the positive moments recommended by the committee. The distribution of the reinforcing bars was, however, quite different from that recommended by this committee.

Although the nominal thickness of the slab ($8\frac{1}{2}$ in.) was less than that required by building regulations, the slab fulfilled the common requirements for compressive and shearing stresses in concrete of the high quality shown in the tests of prisms taken from the slab. The provisions of the Joint Committee on Concrete and Reinforced Concrete for bending moments and working stresses in concrete having a test strength of 3000 lb. per sq. in. give a thickness about the same as the designed thickness of the slab.

In view of the large number of lapped bars not shown on the building plans, the use of high-carbon steel instead of the mild steel specified, and the unexpectedly high strength of the concrete, it is not strange that the floor carried a much higher load than was anticipated when the test was begun.

The test was made as investigative work of the Engineering Experiment Station of the University of Illinois. A more detailed report of the test is being published in a bulletin of the Engineering Experiment Station. Acknowledgment is made for valuable aid received in carrying out the test and in providing funds for expenses connected with it. Among those contributing in this way were the Portland Cement Association, the Universal Portland Cement Co., the Illinois Steel Co., the Pennsylvania Railroad Co., the Chicago Union Station Co., and the Condron Co.

DISCUSSION.

Mr. Godfrey. MR. EDWARD GODFREY (*by letter*).—The results of this test should silence forever the theorist who claims that pouring concrete around steel rods in a flat-slab produces a combination of materials that transcends the laws of ordinary matter, such as the laws of statics and the theory of flexure. The theorist who cuts his bending moment in two because it is a flat slab and then splits it again because the slab takes a saucer shape under a load has no longer a leg to stand on.

The authors tell us that "It may be of interest to note that the sum of the positive and negative moments accounted for by the measured stresses in the reinforcing bars has almost the same value as the sum of the moments recommended by the committee." This refers to the Joint Committee's report recently submitted and criticised as being too conservative by practical flat-slab builders.

The authors further tell us that "It is evident that portions of the floor outside the loaded area contributed measurably to the resistance developed in the slab." Manifestly, then, without the aid supplied by unloaded portions of the floor the sum of the positive and negative moments accounted for by the measured stresses would have been still greater. Without any doubt it would have been just about exactly the theoretic value, that is, the value that it can be proved, by the simplest theory, that it must be. It is a pity that the test was not made, as it could readily have been, in such manner as to eliminate the effect of the surrounding unloaded floor. Of this more will be said later.

John R. Nichols, in Transactions Am. Soc. C. E., 1914, Vol. 77, p. 1670, showed what the writer had pointed out in *Engineering News*, Feb. 29, 1912, Vol. 67, p. 403—namely, that there are perfectly definite static limitations to the bending moments in the main sections of a flat slab. The values of these static limitations are easily deduced by the simplest theory, and their correctness cannot possibly be questioned. The Joint Committee recommends values for the bending moments that are something like 10 per cent less than these static limitations. The results of the test described in this paper demonstrate that moments of this intensity can be expected even when a floor is not loaded in a critical manner.

The writer has said it is a pity this floor was not loaded critically. He would almost say that it is a crime that an opportunity of this sort, for demonstrating once and for all the strength or weakness of a flat slab, should be let slip. The critical loading of a flat-slab floor is a complete band of bays across a building. No other loading can demonstrate anything but a partial truth. No such loading (the critical loading) was ever made on a flat-slab building. At least none has ever been published. I have repeatedly challenged flat-slab builders to make a test of this sort and have called the attention of the profession to the desirability of making such test. No one who reads discussions on flat slabs can be ignorant of this challenge, neither can anyone deny that the only critical test on a flat-slab floor is one that overcomes the tendency of the slabs to assume a dish shape and produces an approximate cylindrical shape instead. **Mr. Godfrey.**

I have demonstrated, even in a beam-and-slab type building, that the difference between loading an interior bay and a line of bays across a building is marvelous. In a flat-slab floor the difference can be expected to be greater.

A TEST OF THE S-M-I SYSTEM OF FLAT-SLAB CONSTRUCTION.

By Edward Smulski.*

The subject of this paper is the test of a slab, designed according to the S-M-I or Smulski System, made by Prof. W. K. Hatt at Purdue University, and the theoretical discussion of the action of rings in resisting bending moments. The main data of the test are taken directly from Professor Hatt's report, for which due credit to him is hereby given.

For the convenience of reference the general data and summary of results are given first. These will be taken up later more fully under proper headings.

General Data and Summary of Results.

Dimensions.	Span	16 x 16 ft.
	Column head	3 x 6 in.
	Thickness of slab	$5\frac{1}{2}$ in. or $\frac{1}{35}$ of the span.†
	Drop panel	6 ft. 3 in. square, 2 in. thick.
	Amount of steel	2.4 lb. per sq. ft.
Design Load.	150 lb. per sq. ft.	
Specification.	Joint Committee moment coefficients.	
Dates.	Slab constructed	Nov. 30, 1916.
	Test began	Jan. 29, 1917.
	Test ended	Mar. 15, 1917.

Maximum Loads Applied. (See Loading Diagram, Fig. 1.)

Unbalanced Load. West panels 950 lb. per sq. ft. for one-half the width of the panel around the center row of columns; 700 lb. per sq. ft. balance west, with brick from aisles piled along their edges. East panels 475 lb. per sq. ft. uniform.

Uniform Load. 700 lb. per sq. ft. with extra brick equivalent to 125 lb. per sq. ft. around column heads (ABCD Fig. 1) to compensate for aisles and observation spaces. Total load 768,280 lb. Area outside column heads 1347 sq. ft.

Design Live-Load (150 lb. per sq. ft.). *Deflection* 0.033 to 0.050 in.

Double Design Live-Load (300 lb.). *Deflection* 0.112 to 0.166 in.

Design Live-Load (150 lb.). *Stresses* 1000 to 3000 lb. per sq. in.

Live-Load (625 lb.). *Stresses* 6180 to 8440 lb. per sq. in.

At a live-load of 650 lb. per sq. ft. the outside columns yielded by cracking at the underside of the column heads and brackets. This changed the flat slab from restrained at the end supports, as contemplated in design, to one more freely supported, increasing stresses and deflections in a ratio described later.

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† The slab was made purposely thinner than used in standard practice so as to make the effect of tension in concrete less pronounced.

In the prefatory note to his report Professor Hatt states as follows: "Investigation involved several elements that have not been found together in previous tests, namely, full loading over the entire slab; load applied over an extended period of time; measurements in winter weather, sometimes at zero temperature; unbalanced loading. The conditions of the test are, therefore, severe compared to those ordinarily made in buildings where only a part of the floor is loaded, and the concrete has had time to harden properly, and where the loads are applied for a few days or a week. The test has an added interest in the action of ring reinforcement in a large scale test panel."

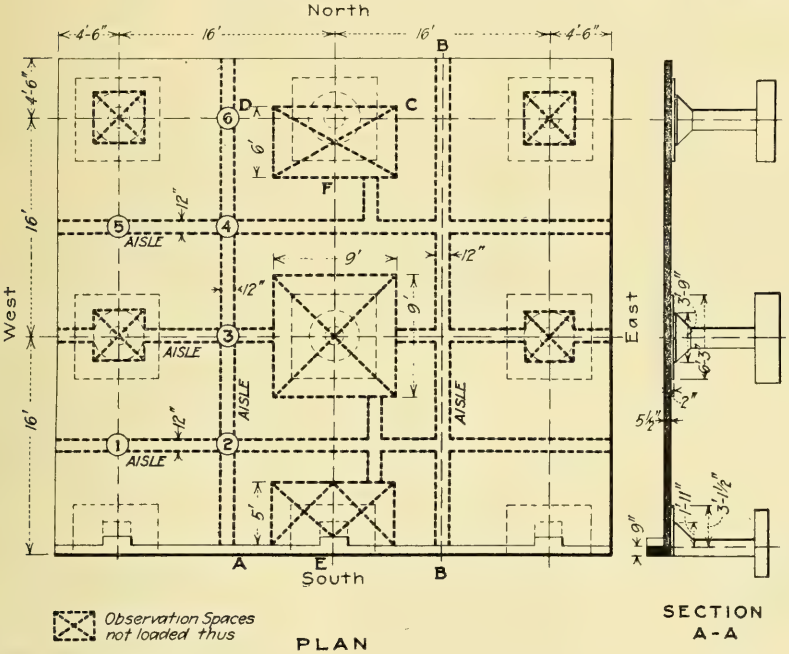

FIG. 1.—GENERAL DIMENSIONS AND LOADING ARRANGEMENTS OF FLAT SLAB.

DESCRIPTION OF TEST SLAB.

The test slab comprised four panels 16 ft. square with 4 ft. 6 in. overhang on three sides and a wall beam on the fourth side. The over-all dimensions were 41 x 36½ ft.

It was intended to make the overhang of the slab long enough to counterbalance the negative bending moment at the column and in that way obtain the same conditions in the north panels as prevail in the interior panels in actual buildings. It is evident from the test results, however, that this pur-

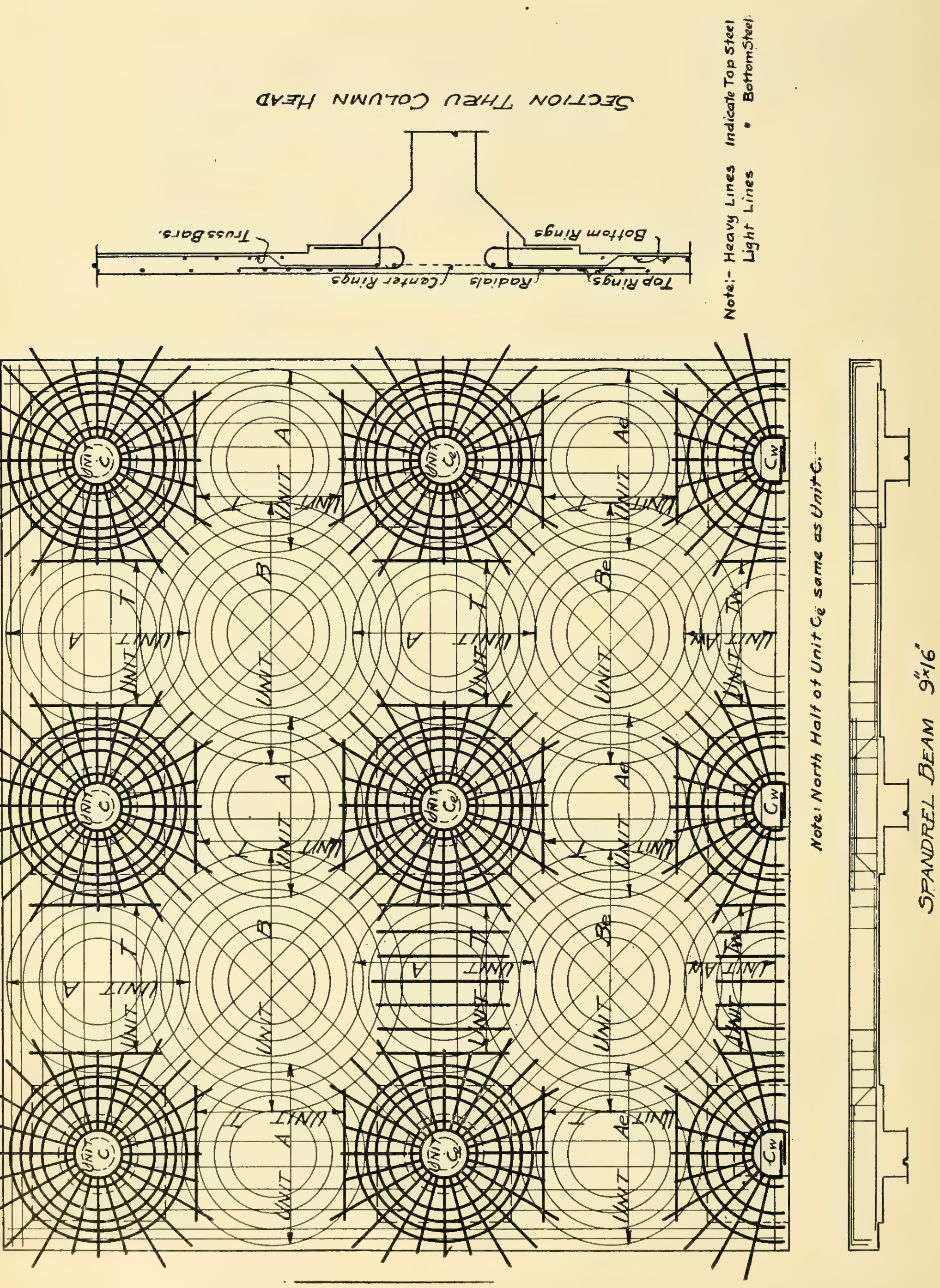

FIG. 2.—DETAILS OF ARRANGEMENTS OF STEEL IN THE SLABS.

pose was not accomplished. The overhang was not long enough so that, due to unbalanced moment, the column heads tipped at high loads producing a condition more severe than in a similar test in an actual building.

Ample footings were provided in stiff gravel soil and there was at no time of the test any indication of settlement of the footings.

DESIGN.

The slab was designed according to the Smulski System by the S-M-I Engineering Co. for a live-load of 150 lb. per sq. ft. Professor Hatt checked the design and found that an amount of steel was supplied as required by the moment coefficients specified by the report of the Joint Committee of July, 1916, in the sections of reference designated by that committee. The two panels adjoining the overhang were treated as interior panels while the other two panels as wall panels. No special provision was made in columns to resist bending moments. The arrangement of reinforcement is clearly shown in Fig. 2.

BRIEF DESCRIPTION OF THE SYSTEM.

As shown in Fig. 2 the reinforcement of a typical panel consists of Units C and T near the top of the slab and Units A and B near the bottom. Units C at the column head are composed of rings and radial bars and Units T consist of straight bars. Units A placed between columns consist of trussed bars and rings.

The system is distinguished from other systems by the use of rings in all parts of the slab and also by the fact that the various units are independent of each other. It should be noted, however, that the different parts of the slab are firmly bound together by the trussed bars and by the overlapping of the units. The reinforcement in Units A and B form a continuous mat extending over the whole slab.

EFFECTIVE STEEL AREAS.

The effective areas of steel in square inches along the sections suggested by the Joint Committee are as follows:

Interior Panel.	
Negative moment reinforcement.	
Column section	4.92 sq. in.
Mid-section	1.3 "
Positive moment reinforcement:	
Inner section	3.00 sq. in.
Outer section	1.94 "
Exterior Panel.	
Negative moment reinforcement:	
Column head section	5.7 "
Mid-section	1.3 "
Positive moment reinforcement:	
Inner section	3.61 sq. in.
Outer section	2.33 "

It should be noted that the south half of the center column head sections had an effective area corresponding to the exterior panel while the north half corresponding to an interior panel.

Observation of stresses were made by Prof. H. H. Scofield of Purdue University with a corps of assistants under the supervision of Prof. Hatt.

MATERIALS OF THE SLAB.

Concrete.—Materials used were Atlas portland cement, donated by the Atlas Portland Cement Co., bank sand and gravel. The mix of the slab concrete was 1:2:4 by measure. The cement was normal in strength. Because of low temperature during hardening the concrete was not up to standard. At the time of the beginning of the test the strength of an 8 x 16-in. cylinder was only 1700 lb. per sq. in.

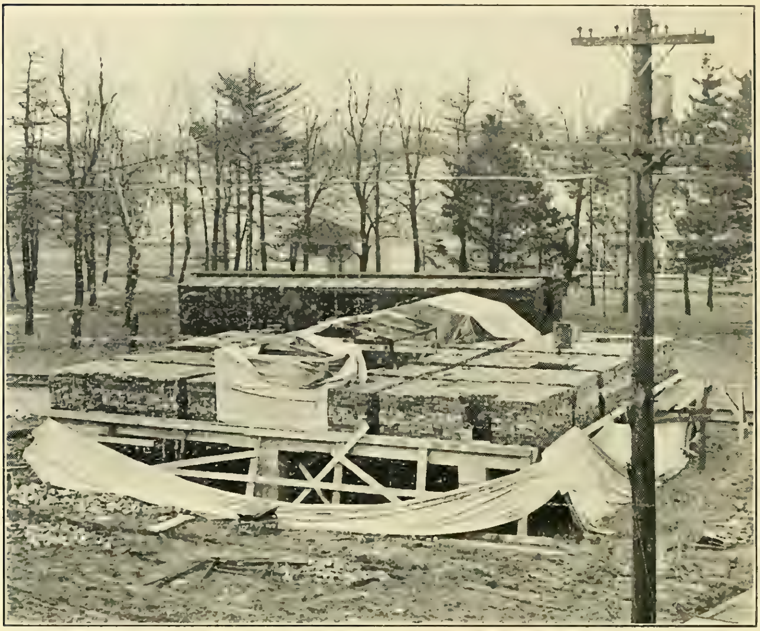

FIG. 3.—VIEW OF TEST LOAD IN PLACE ON THE SLABS.

Steel.—Havemeyer bars of structural steel grade were ordered. By test of steel coupons, however (which took place after preliminary report was issued), it was found that steel was of intermediate grade.

CURING OF SLAB AND AGE AT TIME OF TEST.

The slab was built by Lesley Colvin, contractor, on Nov. 30, 1916, and the concrete artificially heated by salamanders for three days. After this winter temperature prevailed. On Dec. 30 the slab was surrounded with canvas above and below and live steam allowed to escape from the pipes on the bottom and the top of the slab. The steam, however, was turned off at night; the temperature ranged from 80° to 40° F.; consequently the concrete cured very slowly. The test began on Jan. 29 and the observations for final load were made on March 15, so that the test extended over a period of 46 days.

The forms were removed after two weeks, a short time considering the weather conditions.

LOADING.

The loading material consisted of bricks weighing 5.6 lb. each or 25 lb. per sq. ft. per one layer of brick laid flat. To prevent arching of the load towards the supports 12 in. aisles were left running at right angles through the center of panels. With such arrangement any arching would throw the thrust towards the center of the panels producing a condition more severe than uniform loading. For observations vacant spaces were left over the column heads—a square 9 ft. on side over center column, and one-half a square over the middle wall column and the middle edge column along the north-south axis. To compensate for the moment of these unloaded spaces extra bricks were piled around the observation spaces to an amount of 18 per cent.

The loading was distributed over the slab in such a way as to determine the effect of unbalanced loads as well as the effect of uniform loads extending over the slab. The first conditions give the largest positive bending moments and the second the largest negative bending moment.

TABLE SHOWING DEGREES OF TEMPERATURE.

Date.	Temperature, deg. F.	Load (lb. per sq. ft.).	
		West Panel.	East Panel.
January 31	42	150	150
February 3	5	300	300
February 4	2	450	450
February 6	28	625	450
February 7	16	700	375
February 8	10	700 to 950	475
February 10	51	700	475
March 15	34	700	700

The loads of Feb. 6, 7, 8 and 10 were unbalanced and produced severe conditions to the two west panels. Later the load was made uniform over all four panels. The combined effect of long-time loads and unbalanced moment at the corner columns appears in the deflection of the two west panels.

GENERAL RESULTS.

Two phases are distinguished in this test.

First Phase before the strength of exterior columns in flexure was exhausted and before cracks opened in the exterior columns.

Second Phase after definite cracks opened in the exterior columns at the bottom of column heads and the brackets. This phase started at a load from 625 lb. per sq. ft.

The first phase corresponds to the conditions in ordinary building construction.

In the second phase the restraint offered by the connection of the slab and column was destroyed and the slab changed gradually from one restrained

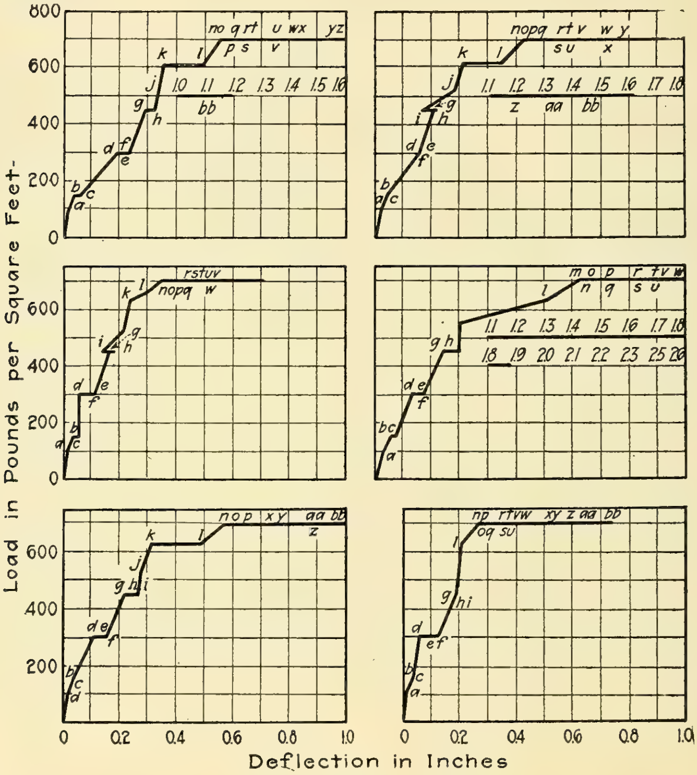

FIG. 4.—DEFLECTION OF SLABS.

at the outside columns to one more nearly freely supported. This increased the deflection and also the bending moments and stresses both in the center and at the interior columns. The ratio of increase is about the same as the ratio between the bending moments in a continuous beam with freely supported ends and those in a continuous beam with fixed ends.

The maximum applied load consisted of: West panels, 950 lb. per sq. ft. around the center column and the center wall column for one-half the width of the panel except for housing spaces; 700 lb. per sq. ft. on the remainder of the panels with brick from aisles piled on the edges of the openings. East panels 475 lb. per sq. ft.

The excess load from the west panels was transferred to the east panels so that the final load consisted of 700 lb. per sq. ft. distributed uniformly over the slab with extra brick equivalent to 125 lb. per sq. ft. around the column head to compensate for aisles and openings left for observation of stresses. (Fig. 1.)

DEFLECTIONS.

Deflections at Design Load of 150 lb.—At design load and at double the design load, or 300 lb., the deflection in inches and in terms of span were:

Uniform load over the slab	150 lb.	300 lb.
At center of panel	0.050 in. or 1/3.840	0.166 in. or 1/1.160
Between columns........	0.033 in.	0.112 in.

Deflections at Higher Loads.—The deflections at higher loads are shown in diagram (Fig. 4). The deflections at 700-lb. load involve the effect of tipping of columns under eccentric load and also the effect of high load applied for long-time. As noticed from the diagram the deflections under maximum applied load increased gradually with the gradual opening of the cracks in the outside columns reaching their maximum in around 12 days at which time the opening of cracks was complete.

LIVE-LOAD STRESSES.

Average live-load stresses in steel are given in Tables 1 and 2.

Design Load of 150 lb. per sq. ft.—The steel deformation for this load were often so small as not to appear significantly on the Berry extensometer in which one division on the dial indicated about 600 lb. per sq. ft. In the rings in a number of cases small compression stresses were observed at low loads, probably due to uncorrected temperature effects, which later changed to tensile stresses. The tensile stresses when indicated were 1000 to 3000 lb. per sq. in. The stresses are therefore quoted in tables as from zero at 150 lb. per sq. in.

At Time of Yielding of Columns.—At about the load of 625 lb. per sq. ft. on west panels and 450 lb. on two east panels the outside columns of the heavier loaded panels began to yield and cracks to open at the underside of column heads and brackets. This load may be considered as the end of the first phase. At this time there were none but fine cracks in the slab and the measured stresses were as follows:—

STRESSES AT 625 LB. ON WEST PANELS AND 450 LB. ON EAST PANELS.

(Zero at 150 lb. per sq. ft. load.)

Unit B. Bottom of Slab.		
Rings 1–2............	6180 interior panel	6220 wall panel
" 3–6............	6680 " "	7420 " "
Truss rods............	7000 " "	7000 " "
Unit C....................	7230 all panels	
Radials..............	8440 inner gage lines	6680 outer gage lines

The above stresses are observed in the heavier loaded panels.

Stresses at Greatest Applied Loads.—At the final readings of steel deformations the slab was in the second phase.

In analyzing the stresses at higher loads it must be remembered that after the load exceeded 625 lb. per sq. ft. the slab entered the second phase in which by gradual opening of the cracks under the brackets and column heads of the outside columns the restraint of the slab was gradually removed. This caused gradual increase in bending moments both in center of slab and at the column and explained why under maximum load the stresses and deflections increased gradually until they reached their maximum after twelve days.

TABLE 1.—SUMMARY OF STRESSES IN LB. PER SQ. FT. TOP OF SLAB.

(Zero at 150 lb. per sq. ft.)

Column.	Position of Gage Lines.		Loading of Panels, lb. per sq. ft.				
	Quadrant.	Between Rings.	West 300 East 300	450 450	625 450	700 950 475	700 700
			RADIALS, UNIT C.				
Center column	Northwest	1–2	3,360	5,760	8,980	33,000	42,800
		2–3	2,460	3,060	7,180	26,400	42,700
		3–4	2,460	....	....	13,200	35,700
		4–5	2,100	....	....	.....	16,800
	Southwest	1–2	3,480	....	8,080	34,200	41,100
		2–3	2,640	2,280	7,240	26,400	31,500
		3–4	1,440	900	3,220	14,400	21,900
		4–5	1,260	....	2,980	.....	12,600
North column	Southwest	1–2	4,860	5,040	10,880	39,600	42,600
		2–3	5,580	6,900	9,700	22,200	48,300
		3–4	3,960	2,400	7,360	24,000	42,600
		4–5	4,240	3,420	7,780	19,800	21,600
	Southeast	1–2	7,260	8,580	6,520	17,400	44,700
		2–3	7,800	7,680	6,460	19,200	40,800
		3–4	4,620	11,340	6,700	19,800	31,800
		4–5	....	5,400	9,280	12,000	36,000
			RINGS, UNIT C.				
Center column	Northwest	Rings 2 to 5	5,460	....	8,110	24,600	40,600
	Southwest		7,000	....	8,040	27,320	38,400
	Southeast		4,900	5,280	5,920	16,140	33,300
	Northeast		5,700	4,800	6,640	21,500	41,940
North column	Southwest		5,300	6,120	8,980	18,780	34,000
	Southeast		4,380	5,600	5,380	13,200	34,000
South column	Northwest		3,480	....	7,560	14,340	28,700

The stresses at the maximum applied load include besides the stresses due to bending moments (*a*) stresses caused by plastic deformations under the heavy load standing on the slab for a long time; (*b*) stresses caused by the change from a slab restrained at wall columns and continuous in all other directions (as contemplated in design) to a slab nearly freely supported at the outside columns.

The maximum measured stresses in lb. per sq. in. were as follows (Zero at 150 lb. per sq. in.):

Unit A	49,800 west panel	23,640 east panel
Unit B—Rings 1–2	33,180 interior panel	13,490 wall panel
" 3–6	42,840 " "	29,600 " "
Unit C—Rings	35,800	
Radials	42,840 inner gage lines to 21,750 outer gage lines	
Unit T	46,250	

TABLE 2.—SUMMARY OF STRESSES IN LB. PER SQ. FT. BOTTOM REINFORCEMENT.

(Zero at 150 lb. per sq. ft.)

Unit.	Position of Gauge Lines.		Loading of Panels, lb. per. sq. ft.				
	Ring No.	Quadrant or Panel.	West 300 East 300	450 450	625 450	700 950 475	700 700
Unit A	1 to 4	Northwest	3,240	6,060	....	14,040	49,800
		Southwest	3,000	5,340	....	43,680	49,800
		Southeast	4,860	....	....	12,580	23,640
Unit Aw		West panel	2,640	....	....	26,240	34,620
		East panel	4,200	....	....	.....	18,840
Units B	3 to 6 (East panels)	Wall panel	3,480	....	....	.....	22,500
		Intermediate panel	5,580	....	7,300	....	34,740
	(West panels)	Wall panel	3,960	6,360	7,420	30,540	36,720
		Intermediate panel	3,900	5,400	6,070	42,120	49,800
	1 to 2 (East)	Wall panel	2,820	....	....	.....	10,440
		Intermediate panel	3,420	....	....	.....	30,000
	(West)	Wall panel	1,680	2,640	6,220	14,280	16,450
		Intermediate panel	3,120	....	7,180	26,580	36,360

Note that at final load the West panels which were subjected to the heavy unbalanced load show much higher stresses than the East panels.

TABLE 3.—SUMMARY OF COMPRESSIVE STRESSES IN LB. PER SQ. FT.

(Zero at 150 lb. per sq. ft.)

Column.	Position.	Loading of Panels, lb. per sq. ft.				
		West 300 East 300	450 450	625 450	700 950 475	700 700
Center column	Near column cap	680	1,210	1,020	1,950	2,300
	Edge of drop	580	920	760	1,230	1,230
	Steel in Compression	3,720	7,440	....	16,420	17,420
Wall column	Near column	490	680	460	1,120	1,280
	Edge of drop	390	530	260	N 390 NW 930	N 650 NW 1,240
West column	Column cap	510	920	770	1,680	2,300
	Edge of drop	390	590	340	.....	660

NOTE.—Stresses in basis of E = 3,300,000 for low stresses and for higher between 20 divisions and 60 divisions on basis of parabola with vertex at 2490.

COMPRESSION IN CONCRETE.

The compressive stresses (see Table 3) in this rather thin slab may be expected to be large. At the greatest load a compression failure occurred in the concrete at the edge of the column cap of the center column in the northwest quadrant with indications of a diagonal tension failure induced by shear.

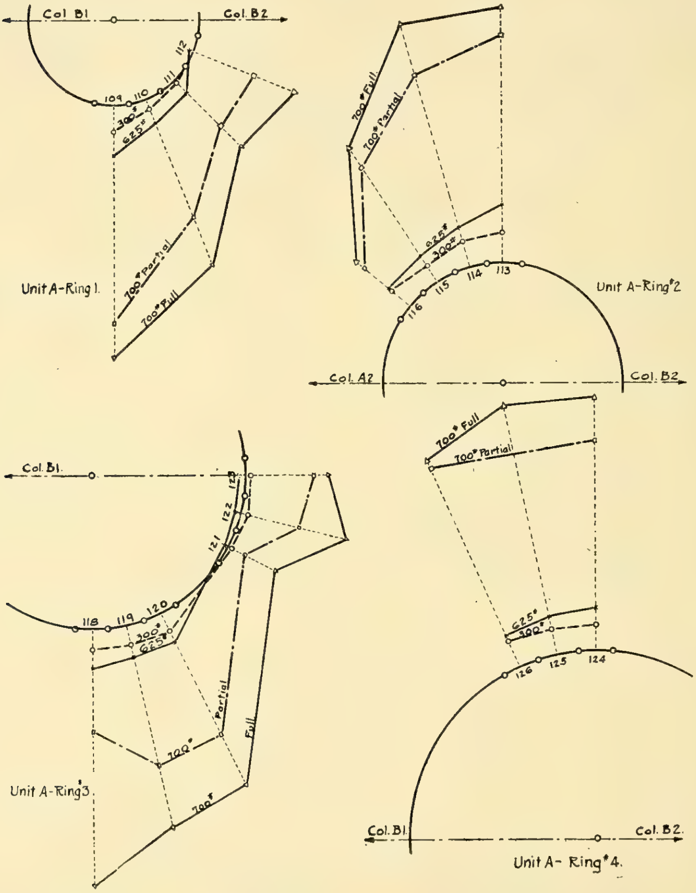

FIG. 5.—DISTRIBUTION OF STRESS IN RINGS IN UNIT A.

VARIATION OF STRESSES IN RINGS OF UNIT A AND B.

Unit A.—The test demonstrates that the Unit A was abundantly reinforced. The cracks across the rings of Unit A appeared at later loads than in other portions of the slab.* Indeed the concrete in the lower portion of the slab between center column and east column enclosed by ring A was practically intact at the greatest loading. (For distribution of stresses in rings see Figs. 5 and 6.)

Unit B.—The hoop action of the center rings of Unit B at the middle of the panel is evident because the circular cracks run around the outside of these rings.

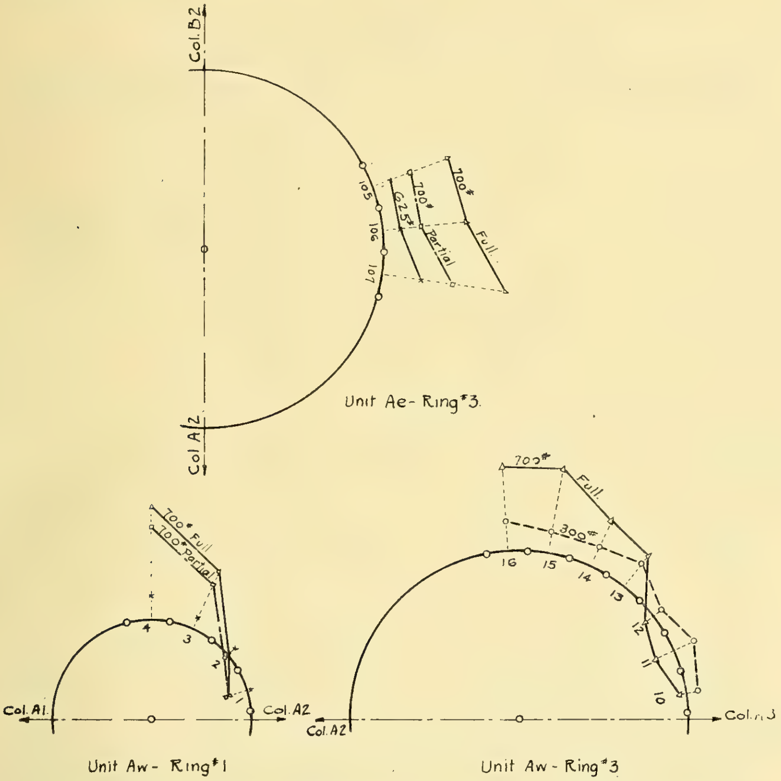

FIG. 6.—DISTRIBUTION OF STRESS IN RINGS IN UNITS A_e AND A_W.

The large outer rings in Unit B are not under uniform stresses because they come into regions of low tensile deformations near the points of inflection and pass to regions of high tensile deformations near the center of the span. They act, therefore, as tension reinforcement between the columns. For distribution of stresses in rings see Figs. 7 and 8.

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* In designing, 60 per cent of the total positive reinforcement was placed between the columns and the remainder in the inner sections. The test indicates that it would be more advantageous to place a smaller percentage in Unit A and increase the amount in Unit B.

STRESSES IN UNIT C.

The stresses in radials are maximum at the gage lines nearest the column head and decrease for the gage lines farther away from the column head.

Stresses in all outside rings in Unit C were fairly uniform and uniformly distributed along the circumference. The hoop action rings is evident. The distribution of steel seems satisfactory. For distribution of rings see Fig. 9.

POINTS OF INFLECTION.

The distance of the points of inflection are given in the table below.

Rectangular direction.................................Span = 16 ft. 0 in. Clear span = 12 ft. 3 in
Diagonal direction.....................................Span = 22 ft. 6 in. Clear span = 19 ft. 0 in

Position.	Direction.	Distance of Points of Inflection.			
		From Center of Column.		From Edge of Column Head.	
		Ft. In.	In Terms of Span.	Ft. In.	In terms of Clear Span.
Center column...............	North......	3 10	0.24	1 10	0.15
	Northwest..	5 2	0.23	3 2	0.16
	South......	4 2	0.21	2 4	0.20
	Southwest..	5 8	0.26	3 10	0.20
West center column...........	East........	4 10	0.30	3 0	0.24*
	Northeast...	5 3	0.25	3 5	0.18
Northwest column...........	Southeast...	5 0	0.22	3 2	0.17
Center wall column...........	North......	4 1	0.26	2 0	0.17
	Northwest..	5 6	0.25	4 3	0.22
Southeast wall column........	Northwest..	5 3	0.24	4 0	0.21

* Doubtful.

CRACKS IN THE SLAB.

At a uniform load of 150 lb. per sq. ft. two very fine cracks developed at the bottom in one panel only, i. e., in the northwest panel.

At a uniform load of 300 lb. except for two fine cracks in the neighborhood of the inner rings in the bottom at the center of the panel the slab was intact. There were no visible cracks at the top of the slab.

At a load of 625 lb. on the west panels and 450 lb. on the east panels the cracks observed at 300 lb. in the center of the panel extended along the whole circumference of the rings. Also fine cracks appeared at the center column head.

Under the load of 700 lb. on the two west panels and 575 lb. on the two east panels a careful survey disclosed only fine cracks, the size of which agreed with the low stresses obtained by the deformation readings. (See Fig. 10–11.) After the load of 950 lb. has been placed on the slab around the center column and center wall column the columns began to yield which caused excessive deformations. The consequent large stretch in steel produced large open cracks.

At the final load which consisted of 700 lb. placed over the entire loaded portion of the test slab with an excess of 125 lb. over the center portion the slab was quite generally cracked. One crack at the center column extended in diagonal direction and may be attributed to diagonal tension.

It is interesting to notice that comparative freedom from cracks across Unit A which indicates that the slab there was over-reinforced in comparison with other parts. The cracks in this portion developed much later than in the middle of the slab. In some portions of the slab, i. e., between the center column and the east column cracks across Unit A were very small even at the final load.

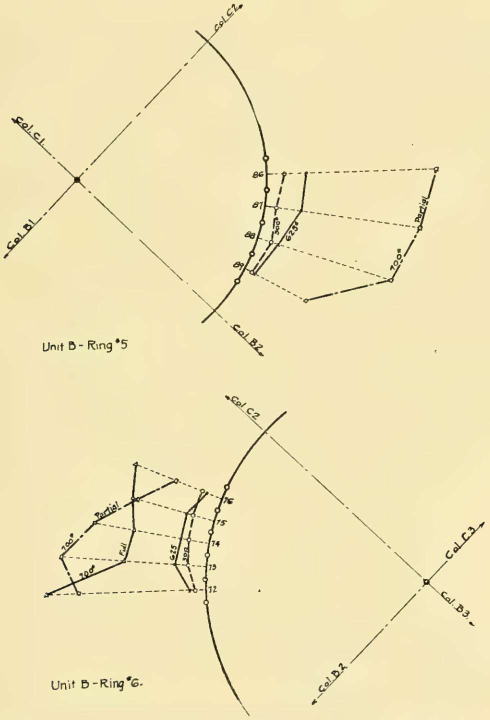

FIG. 7.—DISTRIBUTION OF STRESS IN RINGS IN UNIT B.

In analyzing the results it should be noted that the two west panels were subjected to most severe test. The load was applied before the concrete was sufficiently cured and the column head yielded very pronouncedly. Also the load on this panel was left for a long time on the slab.

After the removal of part of the load cracks were seen that evidenced torsion in the wall beams.

MEASURING STRESSES AND DEFLECTIONS.

The stresses were determined by measurements of deformations in the steel and the concrete by the use of the extensometer devised by Prof. Claude Berry of the University of Pennsylvania. The instruments were made of Invar steel and the changes of temperature therefore had little effect upon the length of the instruments.

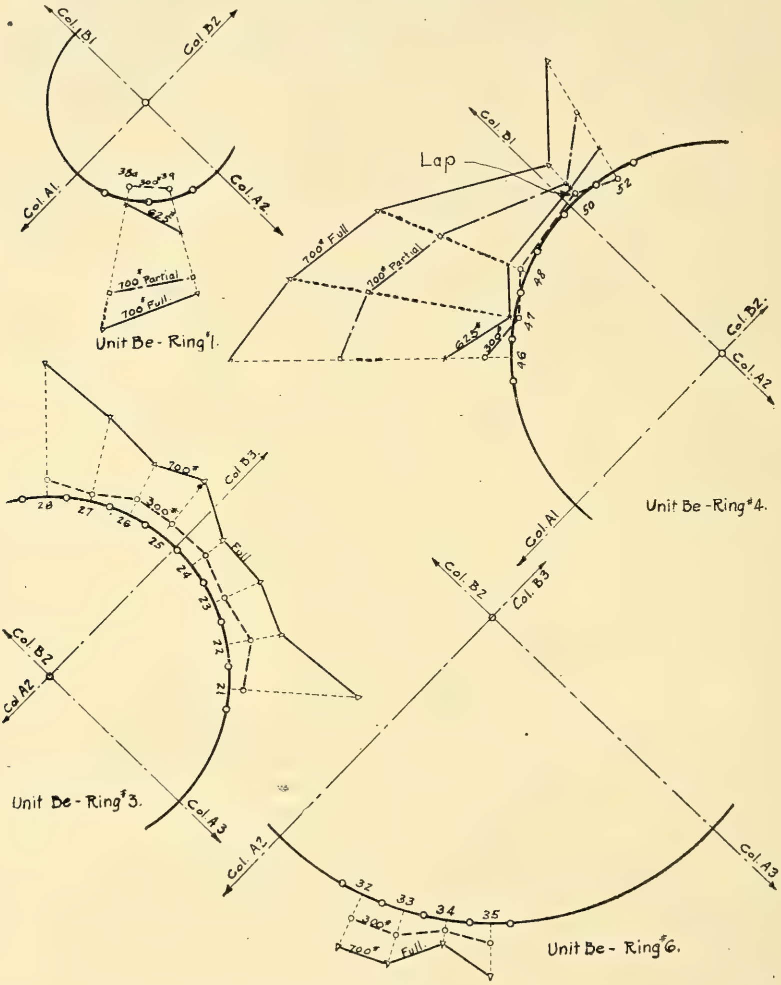

FIG. 8.—DISTRIBUTION OF STRESS IN RINGS IN UNIT B_e.

To eliminate the effect caused by the changes of temperature in the reinforcing steel, extensometer readings were made at frequent intervals upon an unstressed bar embedded in a concrete prism which was placed on the ground below the slab. Corrections were made to all readings to allow

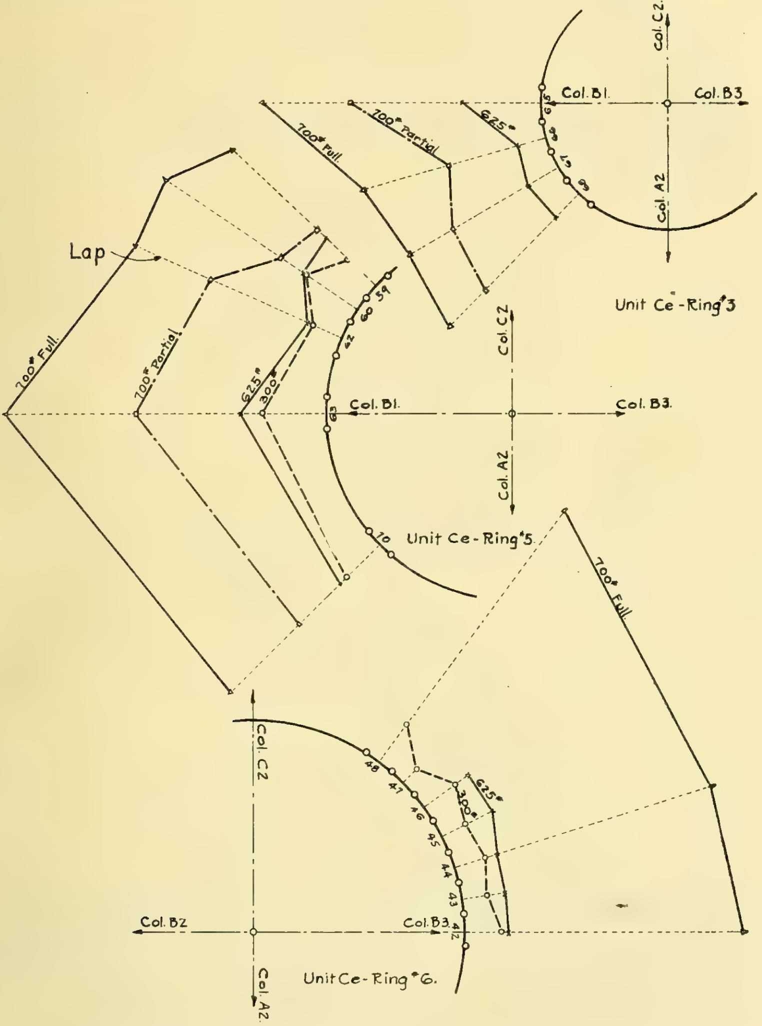

FIG. 9.—DISTRIBUTION OF STRESS IN RINGS IN UNIT C_e.

for the effect of the temperature. These were at the rate of 120 lb. per sq. in. steel for one degree Fahrenheit change in temperature. Some of the observations were made in zero weather.

The number of observation points were as follows:

Bottom steel	254
Top steel	95
Compression	58
Total	407

Deflections were read at the center of panels and between the columns by means of Ames gage rigidly supported on timber framework built for the purpose.

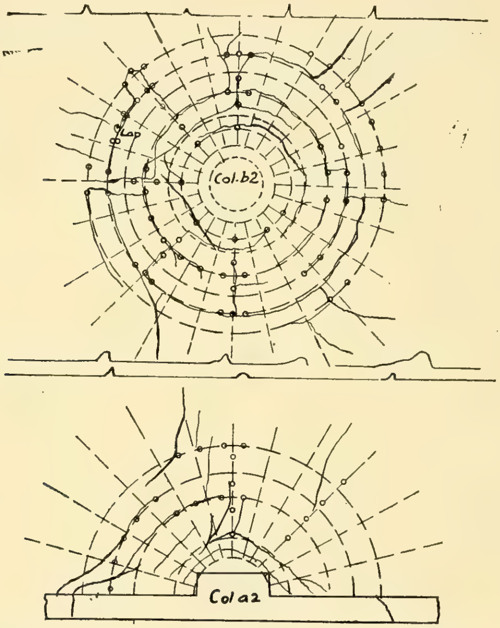

FIG. 10.—CRACKS ON TOP OF SLAB AT FINAL LOAD.

THE STRENGTH OF THE TEST SLAB.

Due to the change of conditions of support at outside column due to the cracks, ultimate strength of the slab could not be determined. The strength of the slab may be inferred from the fact that at an unbalanced load of 625 lb. per sq. ft. on the west panels and 450 lb. on the east panel the slab was comparatively free from cracks and the stresses were below 10,000 lb.

The most severe load applied to the slab consisted of 950 lb. per sq. ft. applied to the area around the center column and center wall column except the housing for half the width of the panel, 700 lb. per sq. ft. with brick from the vacant aisle spaces piled on the edge of the opening applied on the west quarter and 475 lb. per sq. ft. on the east quarter of the test slab. This load

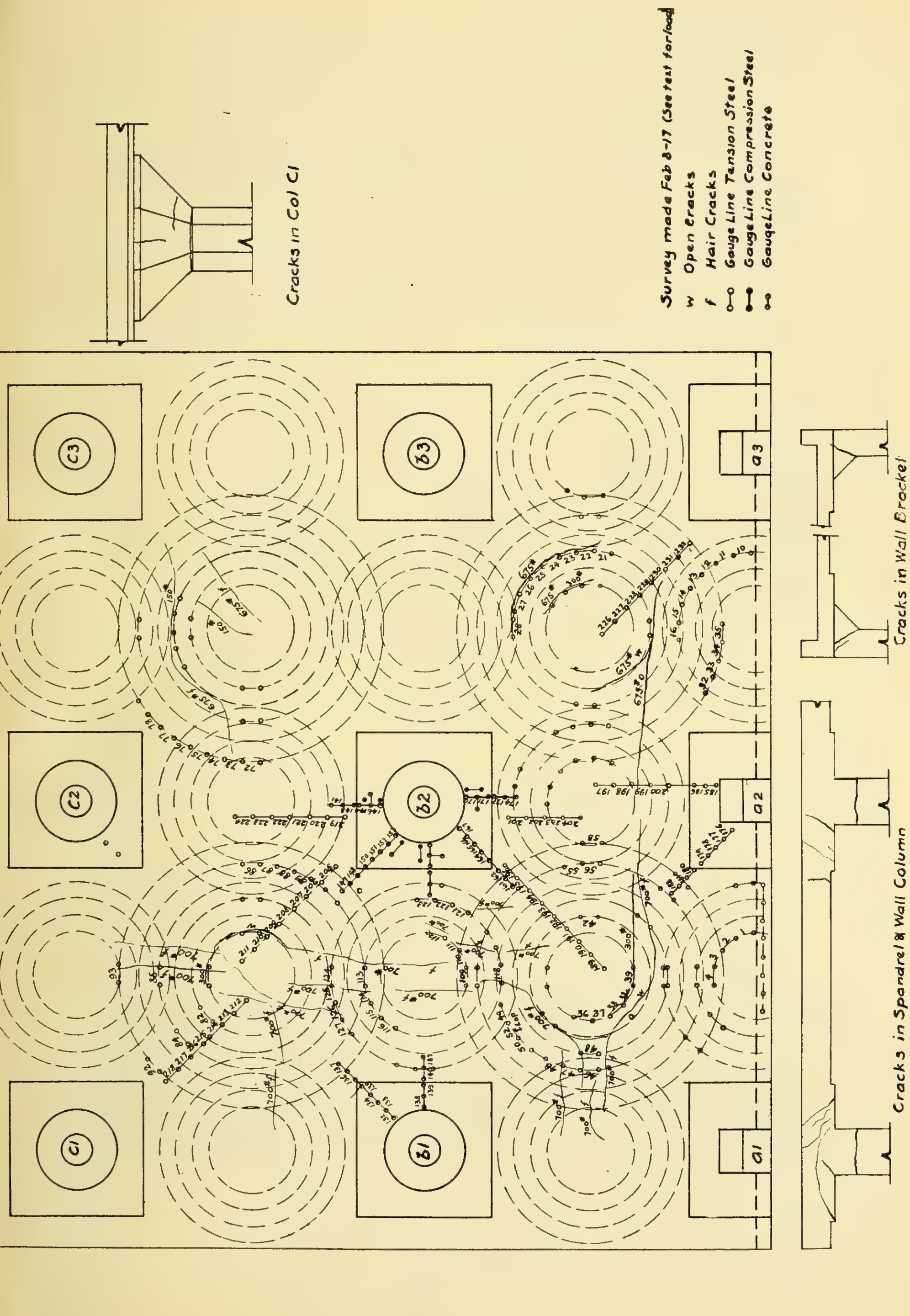

FIG. 11.—GAGE HOLES AND CRACKS ON BOTTOM OF SLAB AT 700-LB. LOAD.

was most severe on the center of the panel because it was unbalanced and caused yielding of the outside columns and thus brought south panels which were designed as interior panels, i. e., fixed at all columns, into a partly restrained condition. In the wall panels the restraint at the columns was almost fully destroyed.

With more favorable weather conditions for curing concrete, stiff columns and larger ratio of thickness of slab to span (all of which items are ordinarily found in actual construction) the strength of the slab would probably have exceeded that reported above.

RELATION BETWEEN STRESSES AND BENDING MOMENTS IN EXTERIOR AND INTERIOR PANELS.

As stated in the report the two south panels were considered in designing as wall panels while the two north panels as interior panels. The amount of steel used in the two south panels, therefore, was increased, as evident from the table of effective steel areas. Attention may be called to the fact that at the center column head the effective area in the southern half was 5.7 sq. in. while in the northern half 4.92 sq. in.

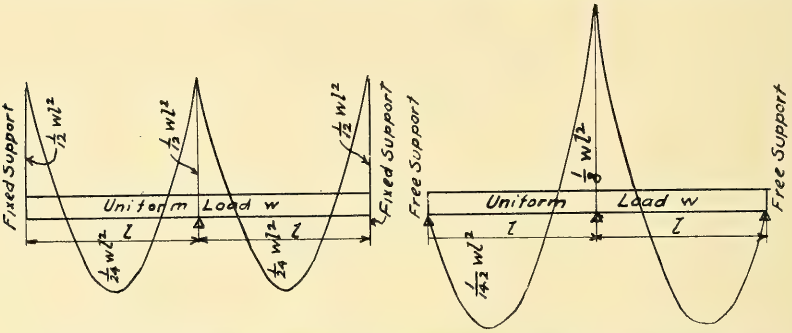

FIG. 12.—RELATION OF BENDING MOMENT FOR VARIOUS CONDITIONS OF END SUPPORT.

From the table of stresses we notice that for small loads the stresses in the interior and exterior panels were about equal. For the final load it is found that the stresses in the wall panels are considerably smaller than in the interior panels. (See tables.)

This fact may be explained as follows:—The overhang provided for the purpose of balancing the loads and producing the condition of fixity at the north columns was not large enough so that under heavy loads the resistance of the columns to bending was exhausted. The column heads, therefore, tipped so that the conditions in the slab instead of being fixed at the supports were only restrained to a small degree. The same occurred at the wall columns. Since, however, the south panels were designed with an enlarged area of steel they suffered through the destruction of restraint much less than the north panels.

THE EFFECT OF CRACKING OF COLUMNS AND TIPPING OF COLUMN HEADS.

The effect of cracks at the columns and the tipping of column heads will be understood if it is kept in mind that by the same the conditions at the supports were modified. After cracking at the southern wall columns the slab became practically simply supported while at the north columns the fixity has been to a great extent destroyed.

The influence of the conditions at the supports on bending moments and stresses may be appreciated by comparing in Fig. 12 the bending moments in a continuous beam of two spans with fixed end supports with the bending moments in a similar beam free at end supports. By comparison it is evident that the change of end conditions increases the bending moments both in the center and at the interior support. The increase in the positive bending moments amounts to 60 per cent while in the negative bending moment 51 per cent.

Table of stresses offers some means of determining of the effect of tipping of the columns. As stated in the report the tipping of columns was much more pronounced in the two west panels, which were loaded with the heavy unbalanced load of 700 to 950 lb. per sq. ft. As a consequence it will be noticed that at the final load, which was uniform over the whole slab the stresses in the east panels (with small cracks in columns) were much smaller than in the corresponding west panels where cracks and tipping was pronounced. A comparison may be obtained from the table below:

STRESSES IN LB. PER SQ. IN. AT A LOAD OF 700 LB. PER SQ. FT.

(Zero at 150 lb. per sq. ft.)

Position.	Direction.	West Panels.	East Panels.	Increase, per cent.
Units A Ring 1 to 4	South	49,800	23,640	110
Unit Aw	South	34,620	18,840	84
Unit B	South	36,720	22,500	63
Ring 3 to 6	North	49,800	34,740	44
Unit B	South	16,450	10,440	62
1 and 2	North	36,360	30,000	20

By inference it may be concluded that if no cracking had occurred in the columns at the east panels the stresses would have been still smaller.

ANALYSIS OF THE SYSTEM.

The various tests of flat slabs, and this test in particular, indicates that a flat slab can be separated along the line of inflection into simple parts as shown in Fig. 13. There are circular cantilevers at the column head; slabs between columns and slabs suspended at four points and forming the central portion of the panel. The distance of the points of inflection from the column head, i. e., points where the bending moments change from positive to negative is treated under separate headings.

In designing the reinforcement of the separate parts may be treated independently and in each of them a sufficient amount of steel provided to resist the particular maximum bending moment to which they may be subjected.

Column Head Section.—At the column head section the portion of the slab within the points of inflection is a circular cantilever fixed around the circumference of the column head and loaded uniformly along the surface and along its circumference by loads transferred to it from the remaining portion of the slab.

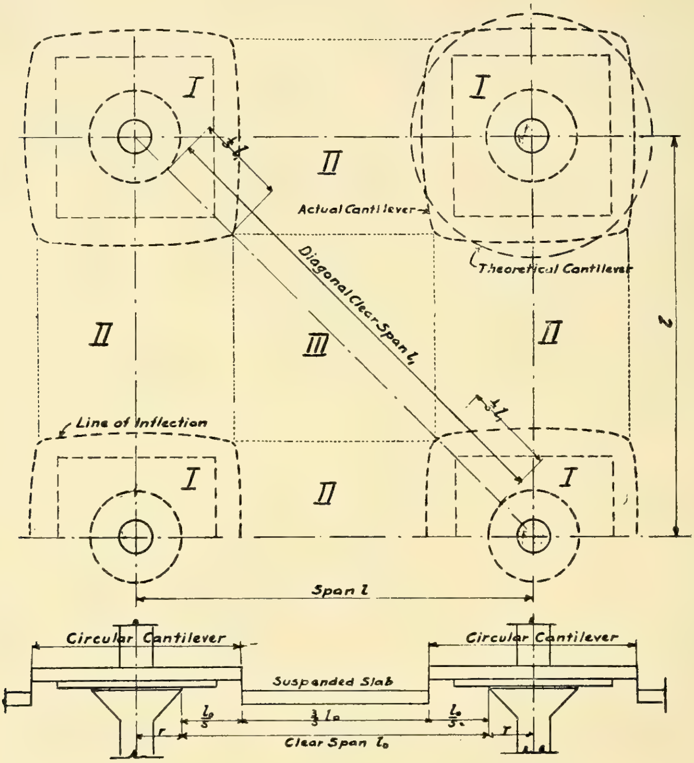

FIG. 13.—FLAT SLAB DIVIDED INTO SIMPLE PARTS.

The direction of the bending moments in this portion must be radial and circumferential as will be explained below. In systems using band reinforcement these moments are not resisted directly but by components. This led to the adaption in various regulations of the moments at sections perpendicular to each other and following the edges of the panel as was intended to conform the direction of the bending moments to the direction of the reinforcement rather than to determine the actual direction and magnitude of the bending moments. As an example may be cited the regulation of the Joint Committee.

That the actual bending moments at the column head act in radial and circumferential directions is evident from the following simple exposition.

Consider small particles of the slab equal distances from the column head forming a closed circle concentric with the columns as shown in Fig. 14. Under stresses each of these particles at the top of the slab will elongate and will assume a position further apart from the column. The radial distance therefore of these particles will be larger than originally. The points formed a continuous ring before deformation. Due to the symmetry of their original position they will also form a continuous curve after deformation. Since the radius in each point increased the particles in the new position will form a larger ring so that each particle must stretch sidewise. The final stresses therefore will be in radial and circumferential directions.

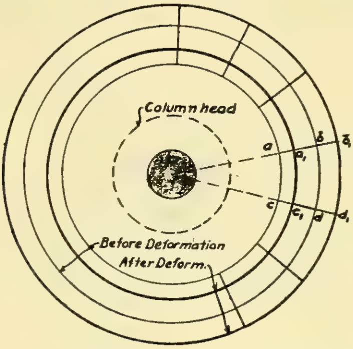

FIG. 14.—DEFORMATION OF PARTICLES.

The same of course can be demonstrated by a theoretical exposition, a good example of which is given in Turneaure and Maurer, "Principles of Reinforced-Concrete Construction," second edition, and Taylor and Thompson's "Concrete Plain and Reinforced," second edition. From the curves and tables in these books and reproduced for a typical case in Fig. 15 it is evident that the radial bending moments are a maximum along the circumference of the column head and decrease in intensity till they reach zero at the points of inflection. The circumferential bending moments act perpendicularly to any radial sections and their intensity varies as shown in Fig. 16.

Slabs Between Columns.—This portion of the slab is subjected to positive bending moment. The principal bending moments in this part act mainly in one direction, which at first is parallel to the edge of the panel and then becomes gradually inclined.

Central Portion of the Slab.—The central portion also subjected to positive bending moments acts like a slab suspended at four corners and loaded with uniform load. The principal bending moments act in diagonal directions.

RINGS RESISTING STRESSES CAUSED BY BENDING MOMENTS.

The use of the rings to resist bending moments has been very often misunderstood and therefore this part will be more fully explained. As indicated above, the conditions of stresses in the three portions of the slab vary. Each of the conditions will be treated separately.

In general consider any portion of the slab one inch wide, subjected to a bending moment. According to ordinary theory of flecture the bending moment will produce internal resisting forces (or stresses) which are compression forces above the neutral axis and tensile forces below the neutral axis. Some of the tensile resisting forces are borne by the concrete but these will not be considered in this discussion as they have no bearing on the subject. The main tensile resisting forces, borne by the steel, act on the level of the reinforcement in the direction of the bending moments.

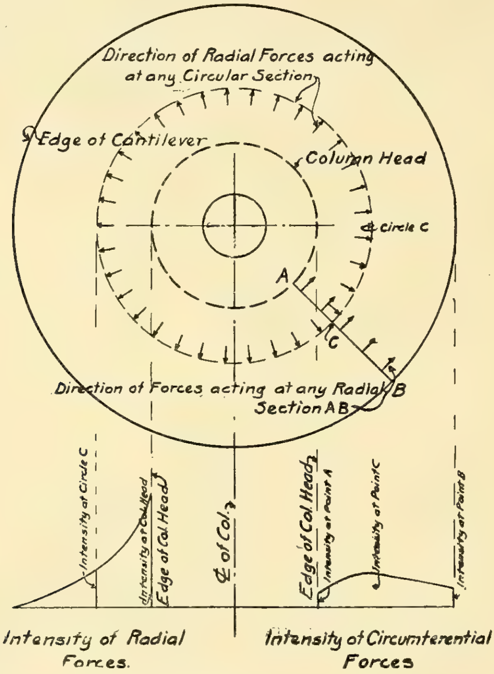

FIG. 15.—BENDING MOMENT AT THE COLUMN HEAD.

The intensity of these resisting forces depends upon the magnitude of the bending moments. For example a slab one inch wide with a moment arm equal to ten inches subjected to a bending moment of 10,000 in.-lb. will have an internal tensile force of 1000 lb. per in. of width acting in the direction of the bending moment.

If these internal forces act within a certain ring they will be resisted by that ring and will produce stresses in the steel bar composing the ring. The reason for it is that the rings are filled with solid concrete which governs their shape. By their hooping action the rings prevent the spreading and deformation of the concrete caused by the tensile forces. The resisting forces are transmitted through the concrete to the rings partly by bond where the forces act parallel to any portion of the ring and partly by bearing. The problem of determining the stresses in rings for a certain bending moment, therefore, will resolve itself into finding stresses in a ring caused by pressures within the ring acting in certain well known directions in resisting bending moments.

Action of Rings at Column Head.—Consider a layer of slab at the level of reinforcement and take a section along the circumference of the column head. The resisting forces caused by the radial bending moments on the level of the rings will be radial (i. e., perpendicular to the section) as shown in Fig. 16. These forces are resisted by all the outside rings. For the sake of simplicity two rings only are shown in the figure. The same applies, however, irrespective of the number of outside rings. The condition, as far as the action of rings is concerned in this case, is similar to that in a reservoir where the steel rings resist the bursting pressure of the water. The method of finding the stresses in the rings under this condition is given below.

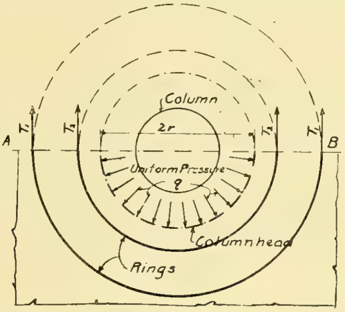

FIG. 16.—ACTION OF RINGS AT COLUMN HEAD.

Assume that the intensity of the forces is q per unit of circumference. To determine the stresses in the rings cut the rings by a section $A\ B$ passing through their center. The removed half of the rings (shown in dotted lines) must be replaced by tensile stresses T_1 and T_2 acting at each section of the rings. For equilibrium the sum of these stresses must be equal to the components perpendicular to the section $A\ B$ of the pressures q. The components can be saeily determined graphically. An accurate determination by calculus is given below.

Since the pressures are uniformly distributed along the circumference of the column head the radius of which is r, the pressure on any arc equals the unit pressure q multiplied by the length of the arc. Lets consider a differential

da of any angle a. The arc corresponding to this differential angle is rda, the pressure acting on the arc if as explained above $qrda$ and its component is $prda\ sina$. From the integral calculus the sum of all the components T from $a=0°$ to $a=180°$ is

$$T=\int_0^{180} qr\ sina\ da=qr\int_0^{180} sina\ da=2rq\ \text{(because}\int_0^{180} sind\ a=2)$$

The sum of all the pressures along one-half of the circumference equals

$$\Sigma q=r\pi q \text{ hence } rq=\frac{1}{\pi}\Sigma q.$$

Substituting this in the above equation we have

$$T=2rq=\frac{2}{\pi 4}\ \Sigma q=0.64\Sigma q.$$

As for equilibrium the sum of components must equal the sum of all the stresses in the rings $T=2\ (T_1+T_2)$ we get

$$2(T_1+T_2)=0.64\ \Sigma q.$$

This means that the stresses in two sections of the rings equal 64 per cent of the sum of all radial pressures acting in one-half the circumference.

The radial pressures are determined from the bending moment, therefore, if M = bending moment per half of circumference, jd = moment arm, A_s = sum of the cross-section s of rings (one section per ring) f_s = unit stress in steel we have $\frac{M}{jd}=\Sigma q$ also $2\ A_sf_s=2(T_1+T_2)$ and therefore

$$2\ A_sf_s=0.64\ \frac{M}{jd}.$$

From the above formula the stresses as well as the required steel area may be easily determined.

Forces Acting Parallel in One Direction.—In the portion of the slab reinforced by units A the bending moments act principally in one direction. The pressures obtained as explained before by dividing the bending moments by the moment arm act practically parallel to each other. The condition in this case is similar to that of a solid disc of concrete with a tight-fitting steel ring subjected to forces acting in one direction. Under the pressure of the concrete, the shape of the ring will change gradually into an oblong curve with the concrete following and always pressing tightly on the ring.

Assume in Fig. 17 a ring filled with concrete of a diameter $2r$ subjected to a uniform pressure q acting along the diameter in the direction of the arrows. The pressure as we know is produced by the bending moment on the section $A\ B$. Now assume the dotted portion of the ring removed and the removed portion replaced by stresses in the ring. For equilibrium the sum of the stresses in both sections of the ring must equal the sum of the pressures. If A_s is area of the bar forming the ring, f_s is unit stress we have

$$2\ A_sf_s=2\ rq$$

If M_1 = bending moment on portion covered by the rings jd = moment arm, A_s = area of one section of rings, f_s = unit stress in steel we have

$$\frac{M_1}{jd} = 2\ rq \text{ consequently}$$

$$2\ A_s f_s = \frac{M_1}{jd}.$$

From the above formula the maximum stresses can be determined.

The stresses in any part of the ring can be found as follows: Consider any section CC_1. Connecting the point C with the center of the ring O we get an angle α with a perpendicular to the section. The length CC_1 then equals $2r\ sin\alpha$ and the pressure on this length equal $2\ rq\ sin\alpha$. For equilibrium the pressures must be resisted by reactions at C and C_1 parallel to the pressures, each of which is equal to one-half of the pressure $rq\ sin\alpha$. To get the tensile and shearing stresses in the rings the reaction will be resolved into tangential and radial components. The tangential component then is the tensile stress in the ring T while the radial component is the shear.

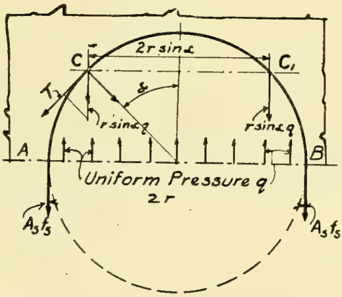

FIG. 17.—STRESSES ACTING IN ONE DIRECTION.

From the resolution of forces we find that the tensile stress T at any point equals

$$T = rq\ sin\alpha\ sin\alpha = rq\ sin^2\alpha.$$

To find the tensile stress at any point in the ring determine the angle α and use the above equation.

Central Portion of the Slab.—As explained under proper heading the pressures in this portion act practically in diagonal directions as shown in Fig. 18.

Consider the stresses for each set of pressures separately, and then add the results. Under one set of pressures the conditions would be the same as explained in the preceding paragraph and the tensile stress at any point would be

$$T_1 = rq\ sin_2\alpha$$

where α is the angle of the radius with a perpendicular to the section $A\ B$.

For the other set of pressures the tensile stress at the same point in the ring is

$$T_2 = rq\ sin^2\alpha$$

where α_1 is the angle of the radius with a perpendicular to the other section $C\ D$.

For both sets acting simultaneously the stress in the ring at any point would be

$$T = T_1 + T_2 = rq\ (sin^2\alpha + sin^2\alpha_1)$$

Since the sections are perpendicular to each other the angle α_1 may be expressed as $\alpha_1 = 90 + \alpha$ and $sin\alpha_1$ as $-cos\ \alpha_1$ which squared gives $+cos^2\alpha$. Substituting in the above

$$T = rq\ (sin^2\alpha + cos^2\alpha)$$

or since $sin^2\alpha + cos^2\alpha = 1$

$$T = rq.$$

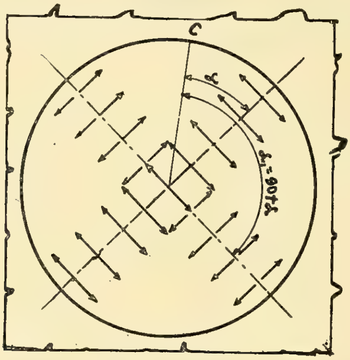

FIG. 18.—STRESSES ACTING AT RIGHT ANGLES.

From the above it is evident that the tensile stress in the rings is the same at any point.

If M_2 is bending moment acting in the distance AB and also at a section perpendicular to AB, jd = moment arm, A_s = area of one section of the rings and f_s = tensile unit stress we have $T = A_s f_s = rq$ and $\frac{M_2}{jd} = 2\ rq$ from which we have

$$A_s f_s = \tfrac{1}{2}\ \frac{M_2}{jd}.$$

From the above formulas we can determine stresses in rings for any conceivable direction of internal forces.

DISCUSSION.

Mr. A. E. Lindau.—Is the loading of 700 lb. per sq. ft., for the entire area, or is it for the area as loaded? **Mr. Lindau.**

Mr. W. K. Hatt.—I may attempt to answer that question. In the maximum load the attempt was made to compensate for rather long openings which it was necessary to make to reach stresses in the exterior range. For that purpose additional bricks were placed on top of the base brick so as to balance the moment. In other words, the moment of the load which was put on, including the extra brick, was intended to be that which would have been present if the slab had been locked up to the top of the corner-head. Of course, in balancing for moment it is not necessary that shear shall be balanced, and so the load that produced the shear would have been present if the entire slab as loaded was deficient, but the intention was to load to balance of moment, 700 lb. per sq. ft. was the total load adopted. **Mr. Hatt.**

One of the interesting features of this test of concrete was in connection with the weather. After Thanksgiving Day the winter weather set in very cold and in order to save the slabs we enclosed them in canvas and piled straw on top and covered that over with canvas. We had steam pipes with perforations in them placed under the canvas and lined along the top of the slabs and similar steam pipes placed below. We turned the live steam in the pipes and allowed it to escape through the openings in the pipes, and so the slab was cared for in this way during a period of about two weeks, with the live steam. A cylinder was placed on top of the slab and a cylinder was placed below the slab and the live steam was allowed to escape. I think the concrete was better than 700 lb. per sq. in., because the compressive stresses around the head of the column were just the same as the other slab constructed later and the action was the same.

One other point that is very interesting to me is the ring reinforcement of the slabs between the columns. This acted as a hooping for the concrete in the center of the panel, retaining it, and therefore the concrete had to pull away that ring and it left it cracked on the outside.

AN APPARATUS FOR THE APPLICATION OF A UNIFORMLY DISTRIBUTED LOADING.

By H. H. Scofield.*

The apparatus here described was devised for the special purpose of testing the efficiency of mesh reinforcement in concrete slabs. The reinforcement was present in relatively low percentages. On account of the nature of such reinforcement, it was thought advisable to have the loading uniformly distributed so that its influence would be the same in all directions.

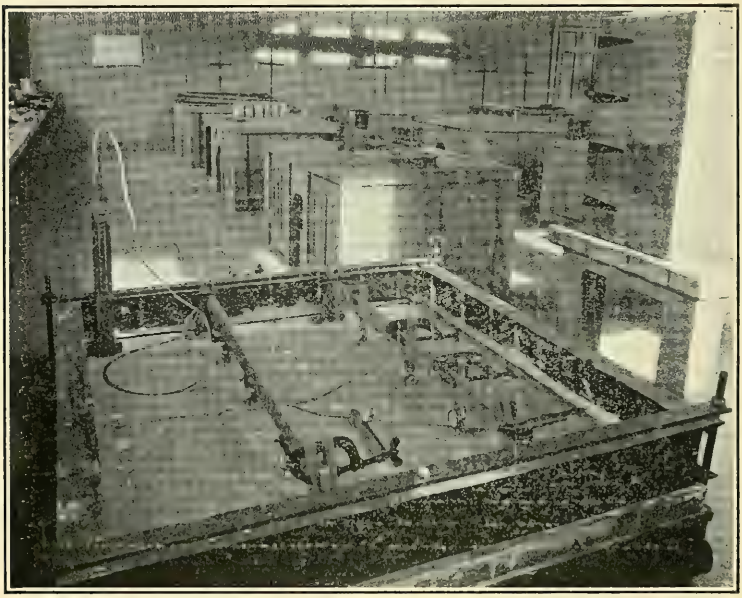

FIG. 1.—APPARATUS AS MOUNTED FOR TEST OF SQUARE SLAB SUPPORTED ON TWO OPPOSITE EDGES.

The apparatus was used with success in this series and later some very good results were obtained in tests of concrete reinforced with various types of metal lath. The tests and results given here, as illustrative of the apparatus and its use, are taken from a senior thesis in the School of Civil Engineering, Purdue University.†

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* Assistant Professor of Civil Engineering, Purdue University, Lafayette, Ind.

† By J. F. Parmer, '16, and R. E. Simpson, '16.

DESCRIPTION OF THE APPARATUS.

The apparatus is shown by photograph in Fig. 1 and by drawing in Fig. 2.

The loading mechanism consists of a square tank five feet on a side and about 6 in. in depth. A loose diaphragm covers the tank and is fastened securely to the four top edges. This air-tight covering was made of a good quality of rubber sheeting. The tank thus covered was connected to a pressure air supply and also to an open tube manometer for measuring the pressure within the tank at any moment.

The test slab was placed across the top of the tank and rested on some interior supports. The upward thrust of load caused by the inflated diaphragm coming up against the slab, was taken by a four-sided I-beam frame, which in turn was held by cross pieces and bolts extending down on the outside and attaching to the I-beam supports of the tank on the bottom.

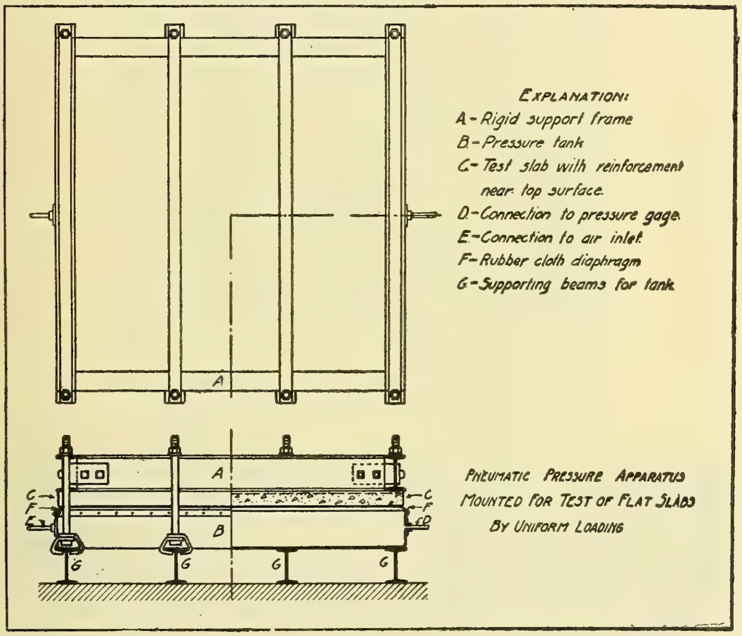
EXPLANATION:
A - Rigid support frame
B - Pressure tank
C - Test slab with reinforcement near top surface.
D - Connection to pressure gage.
E - Connection to air inlet.
F - Rubber cloth diaphragm
G - Supporting beams for tank
A
A
B
C
F
E
C
F
D
G
G
G
G
PNEUMATIC PRESSURE APPARATUS MOUNTED FOR TEST OF FLAT SLABS BY UNIFORM LOADING

FIG. 2.—DETAILS OF PNEUMATIC PRESSURE APPARATUS.

OPERATION.

The operation of the loading was to turn on a supply of air more than that sufficient to break the test specimen. This was allowed to escape through a waste cock. The air pressure in the tank was raised by simply throttling down the waste of air and the result was a very sensitive and uniform control

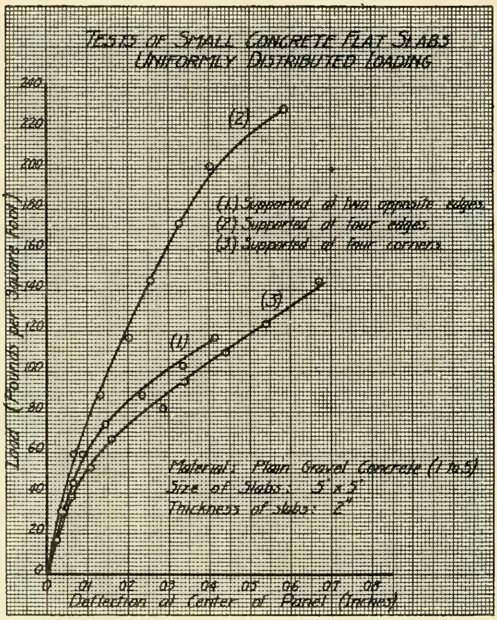

FIG. 3.

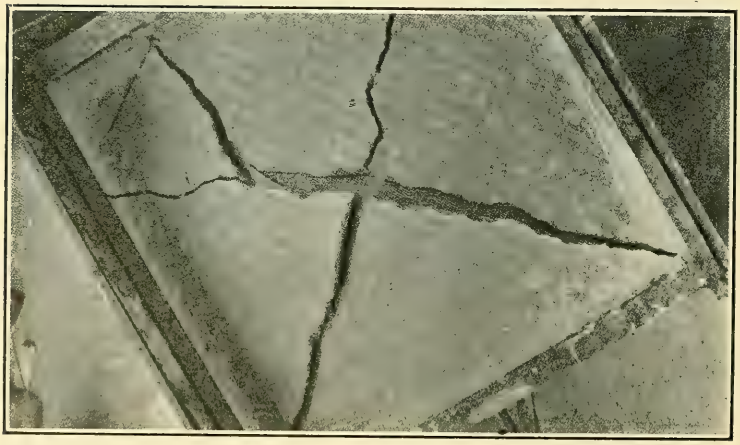

FIG. 4.—FAILURE OF PLAIN CONCRETE SLAB SUPPORTED ON FOUR EDGES.

of the loading. The weight of the test slab itself was eliminated by taking all initial readings when the slab, by itself, was just floating on the rubber sheeting inflated by the air pressure within the tank. The upper supports and bolts were then applied and the loading continued.

RESULTS.

The various results here given are not so given for their value as results, inasmuch as the tests represented under the different conditions are too few and were undertaken mainly for the purpose of studying the apparatus and its action.

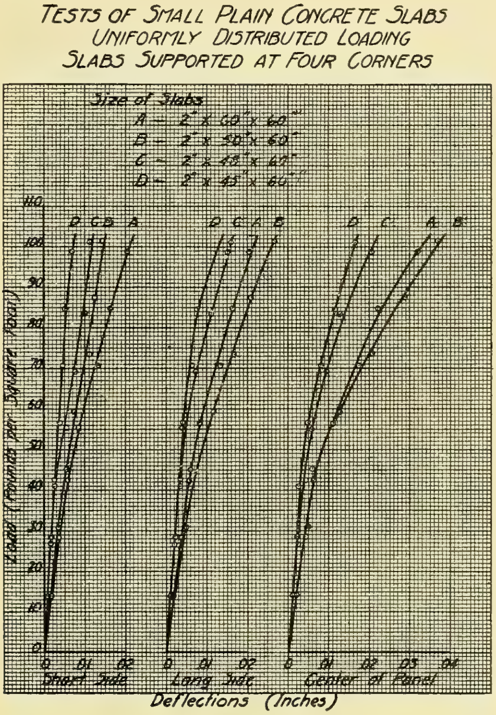

FIG. 5.

The upper or tension surface of the test specimen is open and free, giving ample opportunity for observation of failures and measurement of deformations.

In addition to the tests of mesh reinforcement and metal lath before mentioned, the following series have been undertaken in student thesis work:

(*a*) The relative strength and stiffness of thin square plain concrete slabs when supported on two opposite edges, four edges and four corners.

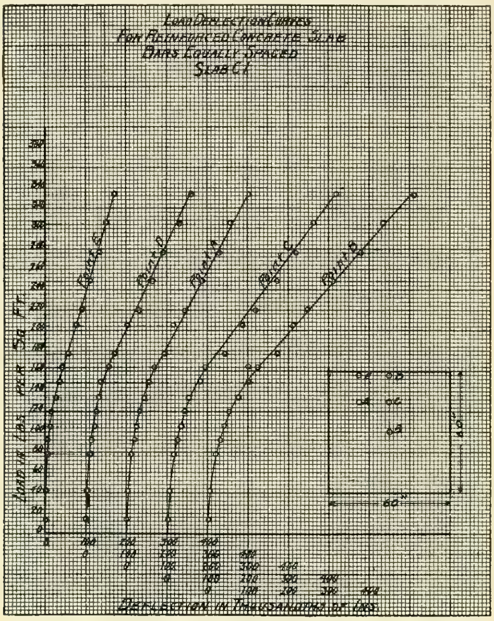
Load Deflection Curves
For Reinforced Concrete Slab
Bars Equally Spaced
Slab C1
Load in Lbs. per Sq. Ft.
Point E
Point D
Point A
Point C
Point B
60"
Deflection in Thousandths of Ins.

FIG. 6.

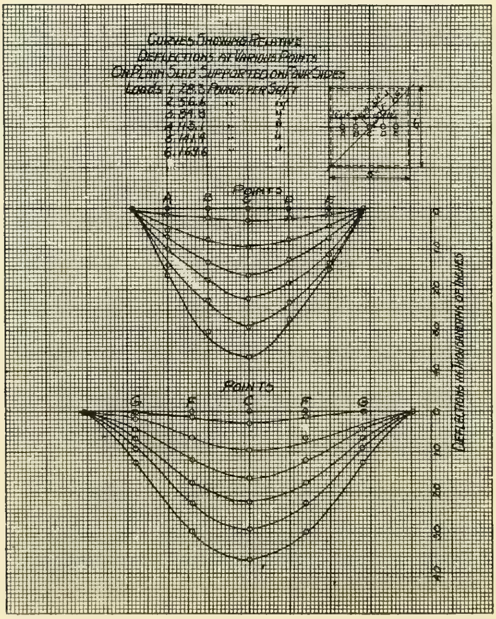
Curves Showing Relative
Deflections at Various Points
On Plain Slab Supported on Four Sides
Loads 1. 28.3 Pounds per Sq. Ft.
2. 56.6
3. 84.9
4. 113.1
5. 141.4
6. 169.8
Points
A
B
C
D
E
Points
G
F
C
F
G
Deflections in Thousandths of Inches
0
10
20
30
40

FIG. 7.

Fig. 3 gives some of the load deflection curves for this set of tests. The conditions for the tests were uniform with the exception of kind of support. Fig. 4 shows a photograph of the manner of failure of the slab supported on four edges.

(*b*) The effect of changing the ratio of length to width of slab on strength and stiffness of small plain concrete slabs supported on four corners. Fig. 5 shows some of the load deflection diagrams for this set of tests.

(*c*) A study of the effect of change of spacing in small reinforced concrete slabs supported at the four corners. Figs. 6 and 7 show diagrams for some of the results obtained from these tests.

In conclusion, it should be said that although the central idea of the apparatus is not new, yet it represents a special adaptation, which might be of interest and possibly of use in the solution of some of the problems of testing, which are continually arising. It is in this spirit that the matter is presented.

ORNAMENTAL AND DECORATIVE CONCRETE.

By R. F. Havlik.*

Decorative concrete may be divided into the following three classes: (1) Concrete garden furniture; (2) Concrete trim stone and decorations for buildings; (3) Decorative mass concrete. The third class is largely one of design and is applicable to any concrete work.

These three divisions include almost all concrete work, except plain concrete used solely for structural purposes, such as in reinforced-concrete buildings and heavy mass work where little or no attention is paid to the decorative feature, and structural strength and economy are the only considerations.

This paper will deal entirely with the first two classes.

The use of concrete for decorative purposes is comparatively recent. While it is true it has been used in isolated instances many years ago, it did not come into general use for this purpose until within the last eight or ten years. On account of the high cost of skilled labor, and also that of natural stone, architects have not sculptured stone for ornamentation as much in recent years as formerly.

With the advent of ornamental concrete, however, it has become possible to furnish highly ornamented stone for buildings at practically the same cost, or slightly more, than that of plain unornamented natural stone. When concrete first came into general use, concrete trim stone was received with great enthusiasm, and was specified for buildings very frequently. This was especially true about twelve years ago. It then experienced a considerable slump, due to many reasons.

One cause for this was the fact that very frequently stone that appeared entirely satisfactory when first produced and put into the building crazed considerably afterwards. Another cause was that many inexperienced contractors attempted to manufacture trim and decorative stone, with the result that the finished product was not as good as expected by the architect. The architect is placed in a very difficult position in recommending and using any new building material, and it is but natural that he should specify those materials which have stood the tests of time, rather than take the risk with new materials. Another cause was that inexperienced masons damaged the stone considerably in handling, principally because they assumed that if damaged the stone could be repaired very easily.

Damaged Surfaces Hard to Repair.

It seems to be human failing that the same care is not used in handling new material that can be repaired readily as is taken with materials that are known to be irreparable, if damaged. Natural stone is handled with exceeding care because mechanics realize that it cannot be repaired satisfactorily. On the other hand every common laborer knows he can make an artificial stone with cement and crushed stone or gravel, and that knowledge causes him to be careless in handling concrete products. As a consequence concrete trim stone was formerly and is still to a great extent carelessly handled in buildings on the supposition that whatever damage is caused in handling can be repaired in the buildings and not show. This can be done when white cement is used for the facing and the surface of the stone is coarse. It has also been done in a few instances with gray cement but it is a much more difficult process, because the color of the stone depends on the amount of water used in the facing, and the same amount must be used in the patching if the repaired portion is to look like the original.

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* Engineer, Mooseheart Institute, Mooseheart, Ill.

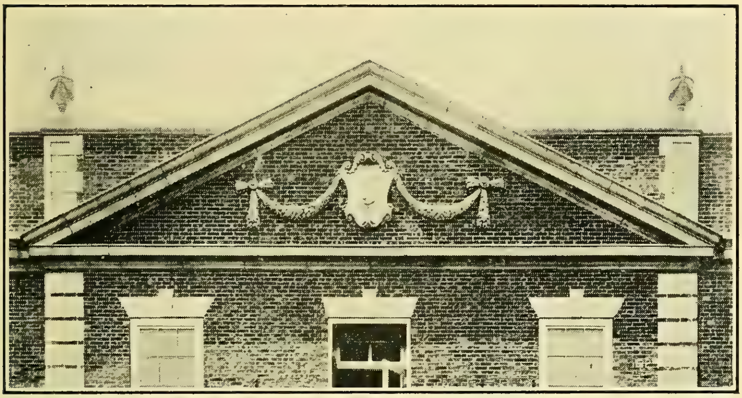

FIG. 1. DETAIL OF CAST CONCRETE FESTOON ON APARTMENT HOUSE.

Many mechanics claim that they can do this but in my experience I have found that few can do it satisfactorily. I have employed a great many men for this kind of work, and of the entire number but one could repair damaged trim stone consistently so it could not be noticed. Easy as it seems, this work requires the greatest skill on the part of the mechanic and no two jobs can be handled in the same way.

The crazing and bad handling of most of the decorative concrete turned out years ago retarded the general use of this material. The architects realized, however, that concrete has many advantages over other materials for decorative purposes, because it lends itself so readily to intricate ornamentation, so they looked for means of overcoming these objections. Manufacturers coöperated also, and the result was that many experiments were made to overcome the crazing by washing or scrubbing off the surface cement with acid or removing it by other means.

CRAZING A COMMON TROUBLE.

In mass work in which no special facing material was used an attractive surface was produced by merely exposing the sand and gravel used in the concrete. In better grade of buildings, however, a smoother surface was required, necessitating the use of finer aggregates. The only available aggregates were sand or finely crushed stone and these were used but did not entirely overcome the crazing feature, probably because they usually contain some dust.

As the demand was created for special facing products some manufacturers produced crushed granite in various sizes. By using the coarser ones very desirable results can be obtained and the crazing prevented. Wherever architects have consented to the coarse aggregates and a coarse texture crazing has been practically eliminated.

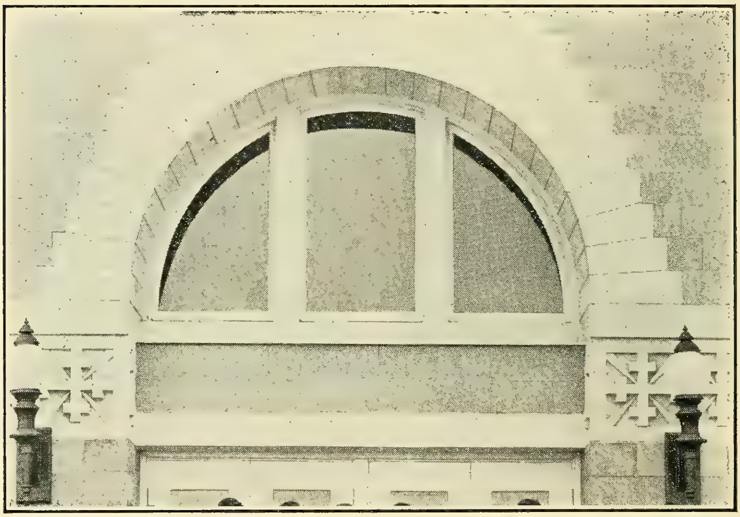

FIG. 2. ARCH OVER DOORWAY OF CONCRETE CAST IN PLACE.

No doubt, there are some manufacturers and contractors that claim they can produce concrete stone with a smooth surface that will not craze, but I feel positive that no one, no matter what his experience, can safely guarantee crazing will not occur or show in concrete stone that has a smooth surface, regardless of the process by which the stone has been made. Even machine tooling does not eliminate this trouble entirely. A few years ago my attention was called to a church in New York City, the exterior of which was built of concrete stone. This had a machine-tooled surface, and I was given to understand had no hair checks, but on close examination I observed many of them. These could not be seen a short distance away from the building, but could be seen clearly by pedestrians passing right by the building. This particular building showed less crazing than most buildings, but this illustration goes to show that even tooling does not prevent it.

My experience convinces me that a reasonably coarse surface is the only sure preventive against crazing. The tiny hair checks may still be there, but the coarse surface hides them.

Many expert opinions have been given on how to prevent crazing or to explain what causes crazing and I have tried out many of them, but found that they did not work out as well in practice as in theory.

In our work at Mooseheart I have obtained the best results by using for facing, aggregates containing no material smaller than about $\frac{1}{16}$ in. in diameter.

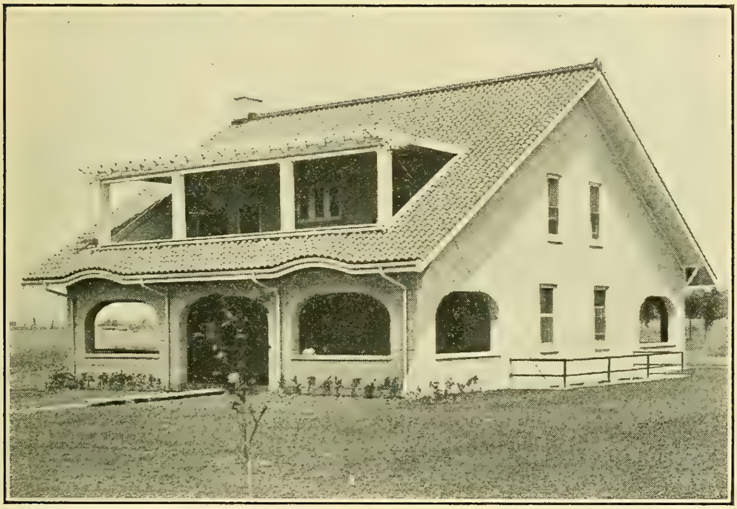

FIG. 3. ARCHES OVER DOORWAYS OF CONCRETE CAST IN PLACE.

Practically every engineer, architect, and manufacturer believes he knows how to prevent crazing or hair checks, and in view of that fact it is all the more surprising that so much concrete trim stone found throughout the country is badly checked. This is practically the only drawback for the common use of concrete trim stone.

In my personal experience I find that architects are favorably inclined toward the use of concrete trim and are quite willing to specify it if they can be reasonably assured of getting a product that will not check. It, therefore, behooves both the architect and manufacturer alike to use those processes and materials which are most likely to obtain these results.

TREAT CONCRETE AS DISTINCTIVE MATERIAL.

If what I have said thus far is true, exposed aggregates and coarse texture are the only safeguards against crazing. It, therefore, becomes necessary to treat concrete as a distinctive building material and not try to use it in imitation of different kinds of building stone.

Concrete stone made of selected aggregates and coarse textures is very pleasing to the eye and the decorative features are purely one of detail and design on the part of the architect. There is some hesitancy in using the coarser textures, but the final result is pleasing beyond all expectations.

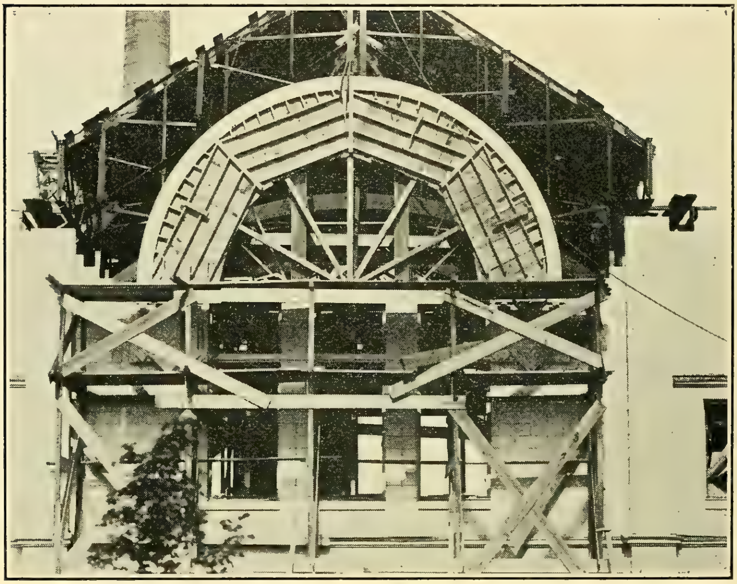

FIG. 4. PLACING CONCRETE ARCH, AFTERWARD MARKED TO RESEMBLE CONCRETE BLOCK WORK.

At Lake Geneva, Wisconsin, is an excellent example of coarse textures in the Northwestern Military Academy which is built entirely of concrete products. The main walls consist of 12 x 24 in. units. The cornices, window sills, and copings were made in wood molds. The columns, about 30 ft. in height, were manufactured in another state and shipped in. All of the other products were manufactured right by the building site. All of this material was faced with white cement and Crown Point Spar which is a crushed light-gray granite. All of the surface cement was scrubbed out of the stone the day after it was made so that the entire surface consists of granite aggregate. No hair checks can be seen. If they are there, they are hidden by the aggregate. In fact, I cannot see how they can exist, except in the joints between the particles of the granite which are infinitesimal in size. The texture used in this is very pleasing and from a short distance the building looks as if it were built of a light gray Vermont granite.

I have dwelt on the subject of crazing at considerable length, but do not apologize because the successful use of decorative concrete stone depends on design and appearance produced by the finished product, and the crazing plays the most important part in the appearance of the finished product. If it occurs it mars the product completely. It is, therefore, important that it be prevented as far as possible.

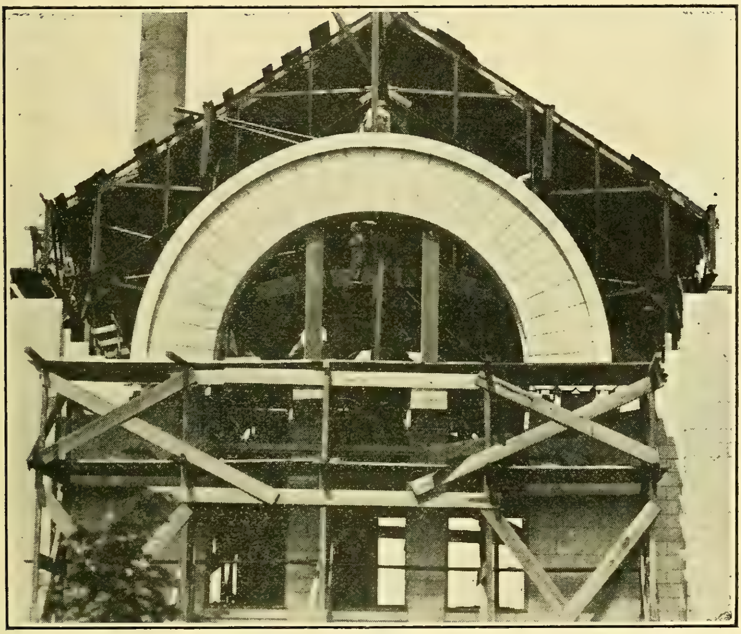

FIG. 5. APPEARANCE OF ARCH SHOWN IN FIG. 4.

I frankly admit that I cannot explain crazing satisfactorily but I do know it does not occur under certain conditions. Some say it will not occur in cast products; others, in dry molded products. I have found it in both. A cast concrete stone that is not treated after it is taken out of the mold will hair check soon after it is exposed to the weather.

SOME PRODUCTS WHICH DID NOT CRAZE.

We have many cast concrete products both in building units and garden furniture, which showed no hair checks after being exposed to the weather for a long period. I have also seen molded products that did not check.

The following facts pertaining to these cases stand out pre-eminently:

The cast products were made in a coarse texture, cured in a saturated steam chamber for not less than three days and up to seven days, were protected from the weather for three or four weeks, and were then dipped in an acid bath until the surface cement was eaten away, exposing the aggregates.

The molded products were sprayed with a fine mist of water immediately after they were made, in order to remove the surface cement from the faced portion. In no case were these touched with a trowel after emerging from the mold.

In some cases, instead of being sprayed with a mist of water right after being made, they were scrubbed the following day with muriatic acid and water and then cured in steam for three days or more, and then protected from the weather for three or four weeks. The curing in a thoroughly moist chamber by means of steam or water seems imperative. Troweling with a steel trowel must be avoided likewise. According to my observations a coarse aggregate containing no dust or very fine material must be used. These three things appear to be essential. Products so made have withstood the elements for several years without showing crazing. In my opinion the crazing or hair checking of ornamental concrete products is the only drawback of architects for their extensive use. It, therefore, behooves the manufacturer to cure his product carefully, protect it from the weather and use coarse aggregates only, for the facing.

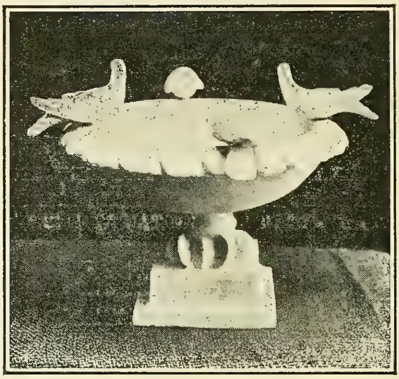

FIG. 6. BIRD BATH OF CAST CONCRETE.

In highly ornamented products containing small sections, it is sometimes necessary to use a finer aggregate, but, in my opinion, none should be used containing dust.

EXAMPLES OF DECORATIVE CONCRETE.

The concrete festoon (Fig. 1) is a part of the Junius Apartments on Michigan Boulevard, Chicago. This particular one measures about 18 ft.

over all. It is not mounted on a block of stone, but the portion tieing it to the main masonry wall is hidden by the festoon itself. It would be very difficult to produce this in stone, but it is comparatively simple in concrete work.

In the Mooseheart school house shown in Fig. 2, there is a large arch over the main entrance. This was to have been made in precast units. The arch is 4 ft. in depth, and some of these units are extremely heavy. In order to avoid damaging the stone unnecessarily it was decided to cast this in place. The outside form was removed a few hours after the arch was cast and the surface was sprayed with a fine spray of water to remove the surface cement. The joints were cut several days after the arch was cast and filled with mortar of the same color as used elsewhere in the building. Visitors, when asked if they noticed anything peculiar about this arch, failed to do so, and expressed surprise that it was made in one piece instead of sections as indicated by the jointing. There is no difficulty in casting a concrete arch. The interesting feature of this is the fact that the form was removed immediately and the surface of the stone finished as it would be in a concrete products plant. That is the special feature of interest in this particular job.

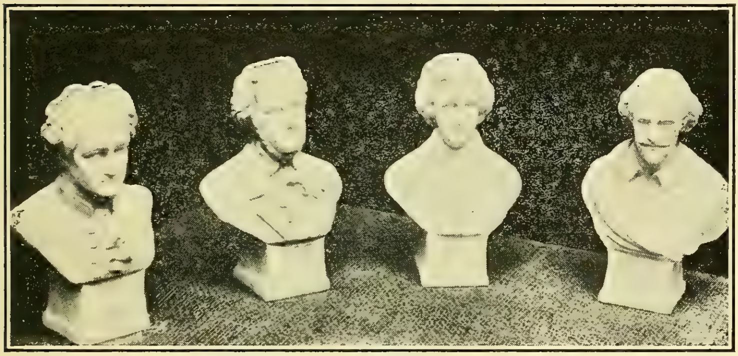

FIG. 7. CAST CONCRETE FIGURES FOUR INCHES HIGH.

David Hall, bungalow type dormitory (Fig. 3), is another illustration of similar work. The arches shown in the porches of this building were also cast in place together with adjoining courses of stone. The joints were cut in several days after the arch was made, and filled with mortar. The outside form was removed a few hours after the arches were cast and the surface sprayed with water. In this instance a coarse granite and white cement were used for the facing. The texture in the arches looks just like the blocks in the balance of the building.

The large arch over the east windows of the auditorium of the Assembly Hall at Mooseheart, Ill., shown in Figs. 4 and 5, was cast in a similar manner. This arch is 17 ft. in diameter and 4 ft. high. This arch like those in the school house and Davis Hall, is surfaced with white cement and granite, and is of special interest on account of this fact. The finished surface is practically a duplicate of the surface of the adjoining blocks made in the concrete products plant. As a rule it is extremely difficult to make a cast unit look like a molded or pressed unit. The two are usually different in texture.

The small vase with doves mounted on the rim (Fig. 6) is of special interest on account of the small size of the objects. So also are the busts of famous men, shown in Fig. 7. These stand about 4 in. in height.

An object of most interest, however, is a small moose mounted on a pedestal. The limbs and parts of the horn are smaller in diameter than an ordinary lead pencil. Each member is reinforced with wire. The entire object is made of white cement and white marble.

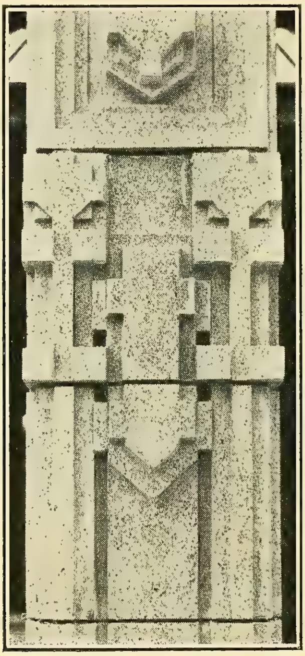

FIG. 8. DETAIL OF PILASTER OF CAST CONCRETE.

I have called particular attention to these small objects as illustrations of how concrete can be used successfully in fine ornamentation requiring minute detail. I have always been a firm believer in the possibility of ornamental concrete and am convinced that architects are ready to use it wherever they can get the material properly treated. As a distinctive building material decorative concrete will prove successful wherever used. Manufacturers producing a satisfactory concrete product can get almost as much for it as is asked for plain stone, and as much or more than is asked for terra-cotta.

In a recent conversation with an architect, he stated that he considered ornamental concrete superior to stone in that it had the advantages for cleanliness that terra-cotta possesses and beauty equal, or superior, to that of stone. This statement, I think, honestly sums up the entire situation. The beauty depends on the architect's detail and the texture used, and the stone can be cleaned readily with acid and water.

In Mooseheart after careful consideration, we decided to teach ornamental concrete work, modeling and molding as a trade to our boys, and this department is now one of the most popular ones in our school. Even some of the girls are taking up this work. Students in this class range in age from twelve to eighteen years.

Highly ornamented concrete can be shipped great distances advantageously on account of the great difference in price between it and terra-cotta, or carved stone, but the plainer ornamented concrete can be made to best advantage at or near the building site.

Architects are ready and eager to use high-grade concrete stone, and conscientious manufacturers can build up a very successful business in this.

FACTORY-MADE REINFORCED-CONCRETE PRODUCTS FOR RAILWAY PURPOSES.

By Charles Gilman.*

The purpose of this paper is to describe the salient features of reinforced-concrete products as applied to railway work, with special reference to those that are made in a well-organized and equipped plant and which, for the sake of brevity, will be styled "factory-made." It is also proposed to show how the development of the factory-made reinforced-concrete product has solved many of the construction, maintenance and economic problems of railway engineering. The merits that determine the adaptability and efficiency of any construction method from the point of view of a railway engineer, and that are distinctly characteristic of the factory-made concrete product, are the following: Permanency as regards both dependability of service and lasting qualities; availability of products to point of installation; portability of product to facilitate transportation and handling; minimum interference with traffic; reduction in the use of motive power and rolling stock for construction purposes; minimum amount of labor used in the field; ultimate economy.

Concrete Pile Trestles.

The first example of factory-made reinforced-concrete products to be considered is the concrete-pile trestle, the development of which began about 1906 by the late C. H. Cartlidge, who was for many years Engineer of Bridges of the Chicago, Burlington & Quincy Ry., and who is considered the pioneer in the use of factory-made concrete piles and slabs in railway work. In a report on "Reinforced-Concrete Trestles," presented before the Western Society of Engineers, April, 1910, Mr. Cartlidge summed up the results to be attained in the following words: "An investigation of the reasons for the great economy of such a construction as the pile trestle shows that it is largely due to the small amount of work necessary to be done in the field. There are no coffer-dams, foundation pits or falsework to be built. Very little raw material has to be unloaded and cared for. The members composing both the substructure and the superstructure are taken out and put in place very largely by machinery with a minimum disturbance of track and delay to traffic. It was evident that if a construction of permanent material having the characteristics mentioned could be devised, the result would be what was wanted."

To meet with the conditions mentioned by Mr. Cartlidge, it is necessary to obtain the maximum efficiency of concrete and steel so as to reduce the weight to a minimum. The reduction of weight is brought about in the

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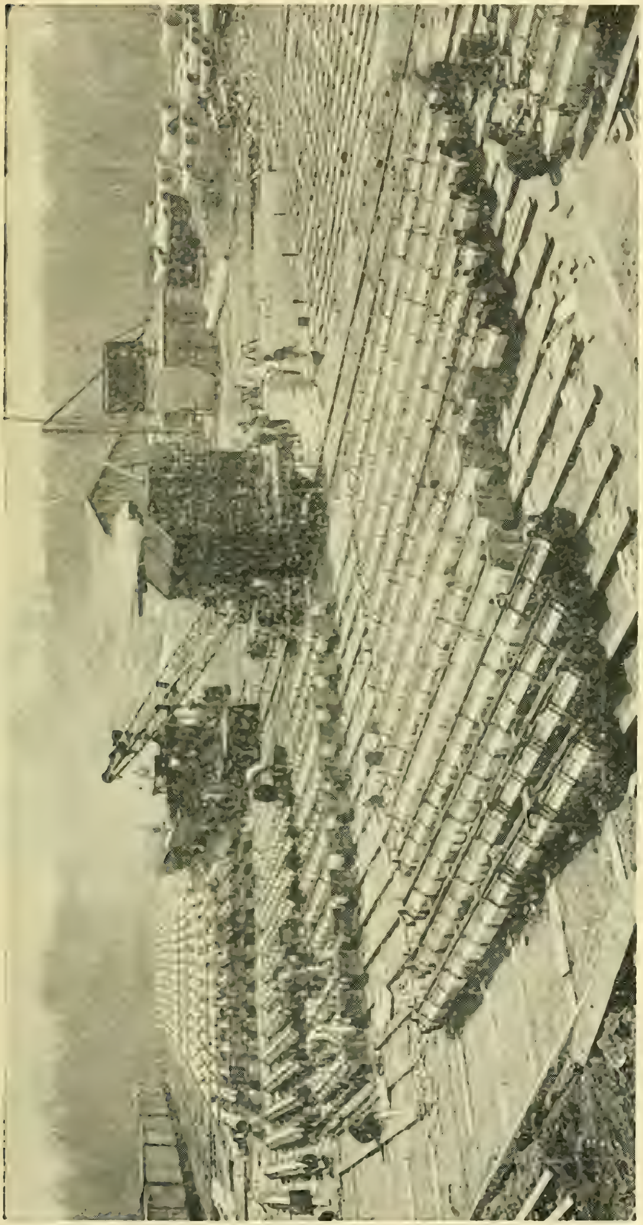


FIG. 1.—VIEW IN YARD WHERE CONCRETE PILES ARE BEING MADE.

design of concrete piles by providing definitely designated points at which the hitches for handling are to be made. The reinforcement can be increased at these points to take care of the stresses developed in handling, rendering it unnecessary to provide for a cantilever action for the entire pile when handled promiscuously. It has been found that piling 50 ft. in length can be economically designed to be handled with one hitch made one-third the way from the butt end; for longer piles, additional hitches should be provided for. The drawing shown (Fig. 3) indicates the general dimensions and system of reinforcement that has been found to produce a very economic type of piling for railway trestles. This pile is calculated to take a working load of 25 tons with a maximum column action above ground of 20 ft.

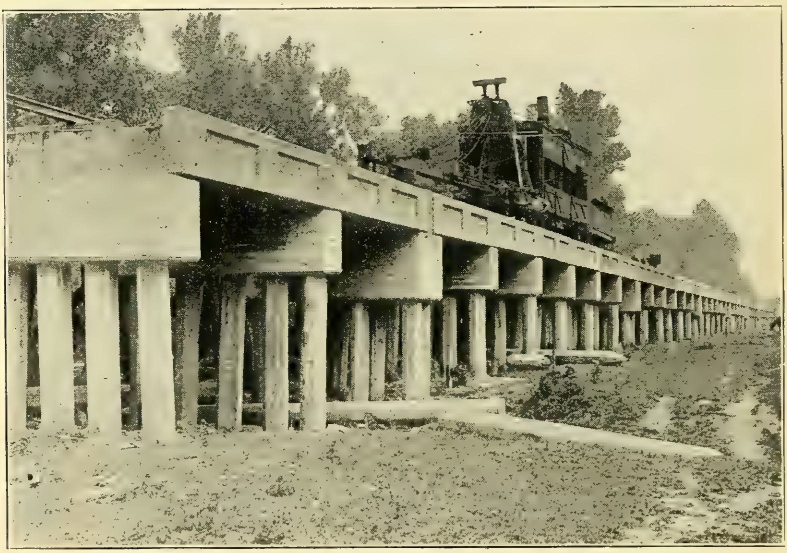


FIG. 2.—TYPE OF TRESTLE BUILT FROM FACTORY-MADE UNITS.

The concrete slab, details of which are shown in the illustration, (Fig. 4) should be designed to provide for easy handling. Two slabs are used for one panel of a single track bridge, as they can be placed with less interference to track and traffic and less derrick power than one slab for the entire width of the bridge. A trestle for one track is from 13 ft. to 15 ft. wide, according to the various railway standards, so that each slab would be 6 ft. 6 in. to 7 ft. 6 in. wide. The length of slab can be standardized to a large extent and still make an economical span. It has been found in practice that spans 14 ft. to 16 ft. from center to center of bents produce the most economic structure for trestles of average height. Lifting stirrups are used for handling and are placed so as to balance the slab transversely. Auxiliary reinforcement is placed in the region of the stirrups to carry the stress that is transferred to the upper surface.

The compact organization of a centralized permanent plant that is possible under the factory-made method renders it practical to make up large quantities of piles and slabs ready for use when needed. When a bridge is up for renewal, the length of piling required can be determined either from the record of piling already driven in former structures, with due regard for the difference in the sectional area of these piles and concrete piles, or by the use of test piles. The usual procedure in the construction of a trestle is to send to the site of the work a pile-driver, generally of the derrick type, with the piling required. This type of driver can be sent either in a work train or in a revenue train, and, having self-propelling facilities, can unload piling, release cars and do the driving without requiring a work train to handle it. As it is usually the custom to work from the old structure that is to be replaced, this driving can be done with the least interference with traffic and without slow orders. After the piling is driven, a small masonry gang casts the caps in place. When the caps have seasoned sufficiently, the slabs are unloaded and set with the derrick. The placing of ballast, the substitution of track ties for bridge ties and the clearing up of the site by the derrick complete the entire construction of the bridge. By comparing this method with the handling of large quantities of raw material, labor, camp equipment, construction equipment and changes in the old structure, or the construction of falsework and the maintaining of slow orders, with the resulting delay to traffic, it can be readily seen that from a railway point of view the factory-made method as applied in this case permits the construction of a fireproof trestle of maximum permanency and ultimate economy.

CULVERT PIPE.

All railway officials owe a great debt of gratitude to Mr. Cartlidge, not only because of the concrete pile trestle, but also, and to a greater degree, because of the adaptation of reinforced-concrete pipe to railway requirements. In 1906, Mr. Cartlidge was confronted with the necessity of replacing many of the original wooden box culverts on his railway. After making a considerable study of the materials available and their relative cost, he decided on the factory-made concrete pipe. In casting about for a design which would be economical and safe as well as distinctive he came across the method of reinforcement which provides for a single line placed in the region of tension throughout the pipe. Instead of distorting the reinforcing cage to place it in the region of tension—which is necessary when using this system in a circular pipe—Mr. Cartlidge left it in repose and changed the contour of the pipe section by inserting, between the upper and lower semi-circles of a circular pipe, tangent distances equal to the thickness of the walls of the pipe. This enabled the reinforcement to take the desired position and still be in repose as a circle. In casting the pipe, the ends of the long diameter were marked "Top" indicating that the long diameter was to be placed

LENGTH.	GROUND LINE TO BUTT.	CLASS 4000.			CLASS 3000			CLASS 2000.			CLASS 1500			CLASS 1000.		
		d	D.	APPROX W'T.	d	D	APPROX WT.	d.	D.	APPROX WT.	d.	D.	APPROX WT.	d	D.	APPROX. WT.
20'-0"	4'-0"	8¼"	13¼"	1720*	7⅝"	12⅝"	1390*	7"	12"	1180*	6⅜"	11⅜"	1080*	5¾"	10¾"	865*
25'-0"	5'-0"	8¼"	14½"	2440*	7⅝"	13⅞"	1940*	7"	13¼"	1670*	6⅜"	12⅝"	1420*	5¾"	12"	1180*
30'-0"	5'-6"	8¼"	15¾"	3310*	7⅝"	15⅛"	2590*	7"	14½"	2230*	6⅜"	13⅞"	2120*	5¾"	13¼"	1550*
35'-0"	6'-0"	8¼"	17"	4340*	7⅝"	16⅜"	3340*	7"	15¾"	2830*	6⅜"	15⅛"	2770*	5¾"	14½"	1950*
40'-0"	6'-6"	8¼"	18¼"	5550*	7⅝"	17⅝"	4230*	7"	17"	3770*	6⅜"	16⅜"	3530*	5¾"	15¾"	2320*
45'-0"	7'-0"	8¼"	19½"	7450*	7⅝"	18⅞"	5230*	7"	18¼"	4680*	6⅜"	17⅝"	4420*	5¾"	17"	2900*
50'-0"	7'-6"	8¼"	20¾"	8400*	7⅝"	20⅛"	6380*	7"	19½"	5750*	6⅜"	18⅞"	5450*	5¾"	18¼"	3440*
55'-0"	8'-0"	8¼"	22"	10000*	7⅝"	21⅜"	7680*	7"	20¾"	6960*	6⅜"	20⅛"	6590*	5¾"	19½"	4070*
60'-0"	8'-6"	8¼"	23¼"	12400*	7⅝"	22⅝"	8800*	7"	22"	8340*	6⅜"	21⅜"	7750*	5¾"	20¾"	4640*

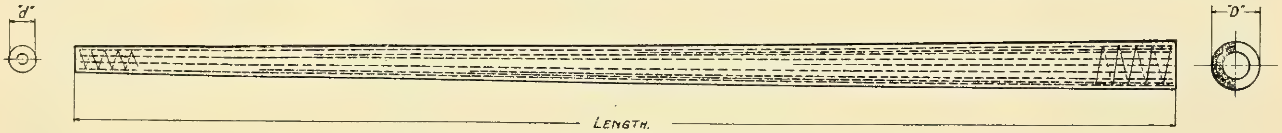


FIG. 3.—SCHEDULE OF CONCRETE POLES HELD IN STOCK.

vertically—the position of maximum strength. This unique design not only produced a pipe of adequate strength and increased flow area, but also afforded an opportunity of proper inspection during and after installation.

In 1907, Prof. A. N. Talbot, of the University of Illinois, made a series of tests on the Cartlidge design of concrete pipe, as well as on cast-iron pipe and other forms of concrete pipe, the results of which were published in Bulletion No. 22 of the University of Illinois. The publication of this report cleared away any uncertainty which existed in the minds of engineers as to the ability of a properly designed and manufactured concrete pipe to successfully carry railway loads and gave the industry the desired impetus.

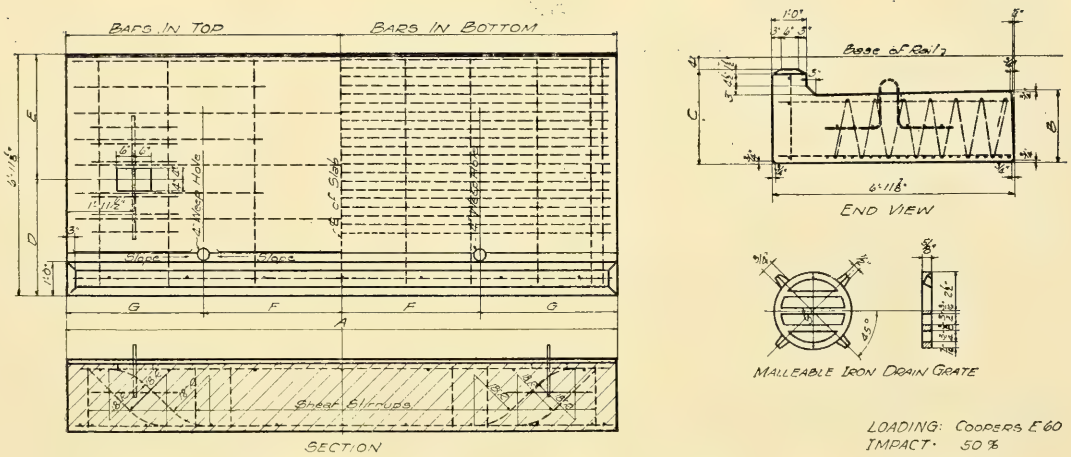
Since 1906 other designs of concrete pipe have been developed and used on railways. The most common of these is the circular pipe reinforced either with a single line all in the region of tension or two concentric lines. To take care of conditions where a pipe of minimum headroom with adequate area of flow is required, a flat base pipe has been designed, the upper section of which is a semi-circle and the invert a curved surface of large radius. This section has the necessary strength, requires practically no foundation under it, and provides a good flow. A design of triangular section has also been developed and used to some extent for low headroom conditions. While this pipe has considerable strength and requires a small amount of reinforcement, it is very heavy on account of the thickness of the walls and has a small flow area.

In designing a culvert pipe, due consideration should be given, on account of handling, to the weight per section by properly proportioning and placing the steel in relation to the thickness of the wall. A culvert pipe should have sufficient strength to carry the usual railway loads with a generous factor of safety so that when subjected to unusual or sudden loads it has the ability to resist. Experience has shown that a concrete culvert pipe should have bell and spigot ends, to give the necessary strength and stiffness at the joint to hold the culvert to grade and it should also be of such a length that it can be loaded, unloaded and installed economically.

GRADE ELIMINATION.

G. E. Tebbetts, Bridge Engineer of the Kansas City Terminal, has worked out a very interesting type of factory-made concrete in connection with the numerous subways and highway bridges that were constructed in Kansas City during the past few years. At first, the abutments, pier foundations, piers and girders were cast in place, using slabs built under the factory-made system. It was then considered feasible to make the girders as units and install them later. A further development included the columns. It is therefore possible to see in Kansas City subways showing the gradual development of the factory-made concrete until now only the abutments and pier foundations are cast in place.

This type of construction has reduced to a very noticeable degree the interference with both railway and street traffic, the time of construction and the ultimate economy of the entire work, to say nothing of obtaining all the advantages of factory-made concrete that have been discussed elsewhere



LOADING: COOPERS E 60
IMPACT· 50%
CONCRETE
SHEAR 40# AND 90#
BOND 100#
COMPRESSION· 650#
STEEL
TENSION· 16,000#.

BENTS	A	B	C	D	E	F	G	WEIGHT OF 1 SLAB
10'0" CTRS	9'11"	1'6½"	2'2½"	3'3⅞"	3'8"	2'6"	2'5½"	17,000 LBS
12'0 "	11'11"	1'8"	2'4"	3'4"	3'7⅞"	3'0"	2'11½"	21,700 "
14'0" "	13'11"	1'10"	2'6"	3'4⅜"	3'7½"	3'0"	3'11½"	27,700 "
16'0" "	15'11"	2'1"	2'9"	3'4½"	3'7⅝"	4'0"	3'11½"	36,000 "
18'0" "	17'11"	2'4"	3'0"	3'4¾"	3'7⅛"	4'6"	4'5½"	45,000 "
20'0" "	19'11"	2'10"	3'3"	3'4½"	3'7⅜"	5'0"	4'11½"	60,500 "

FIG. 4.—REINFORCED-CONCRETE BRIDGE SLABS FOR DELIVERY.

in this paper. The possibility of inspecting each unit of a structure before it goes into service, with the minimum interference of traffic during erection, has made this type of construction very attractive to railway engineers.

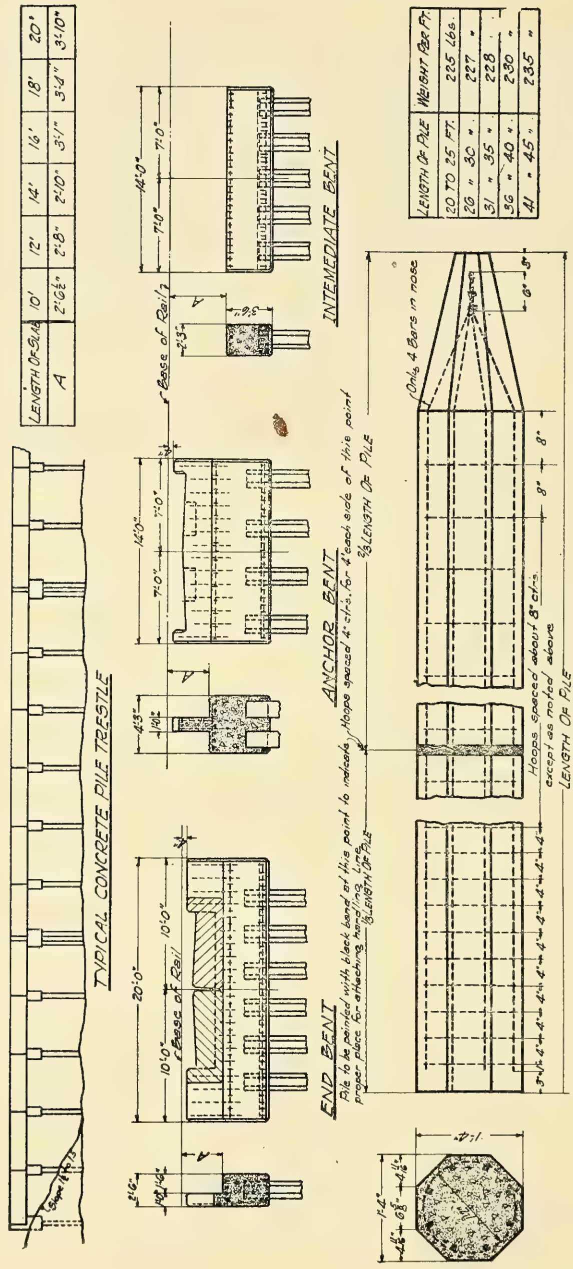
Where streets cross over railways, this same type of unit construction can be used. A design for this particular purpose has been worked out very effectively by A. B. Cohen, Concrete Engineer of the Delaware, Lackawanna & Western Railroad. It is especially effective where the decks of bridges having stone or concrete abutments in good condition require renewal. The use of factory-made beams, girders and hand rails has reduced the field work to a matter of a few hours. Where new foundations for columns or new abutments are necessary, they can be installed without interfering with railway or highway traffic, and the columns, girders and slabs can be set in place by work trains in a surprisingly short time.

HOUSES.

Railway systems require many small houses that should be fireproof, ratproof, portable, sanitary and reasonably safe against malicious damage. The various types used include telephone booths at passing sidings, watchmen's houses at grade crossings, block stations, oil houses, houses for torpedoes and fuses, outhouses, pump houses, scale houses, motor-car houses, transformer houses and cable-test houses. The factory-made product has fulfilled all of the above requirements at a reasonable cost. With the increased use of the factory-made product, it has been possible to standardize many of the types, justifying the expense of steel forms, the use of which has produced accuracy and economical production. Today, the only limitation of the size of houses that can be made in the factory is determined by railway clearances. Particular attention has been given in the design of these houses to permit handling by derricks or by skids. Usually, the roof and its connection to the side walls are so reinforced that a timber under the ridge pole or at the base of the roof fastened to a line through the ventilator or chimney hole will permit handling with a derrick. In most cases the matter of foundation can be taken care of with a bed of cinders or gravel. Where it is necessary to install heavier foundations, particularly where the ground is soft, or to make up for the inequality in the level of the ground surface, as on the sides of embankments, pedestals can be installed at small expense.

MANHOLES.

The construction of underground conduit work by the telephone, telegraph and signal departments of railways has in many cases been materially benefited by the use of portable concrete manholes. Instead of building expensive brick and concrete work in place, which must be protected from the vibration of passing trains in order to season properly, it is possible to place the factory-made product with a derrick in holes which are prepared just in advance of the installation. This reduces to a minimum the time necessary to keep excavations adjacent to the track open, and consequently slow orders. The use of the factory-made manhole had produced not only a permanent, waterproof structure, but it has also materially reduced the cost.



LENGTH OF SLAB	10'	12'	14'	16'	18'	20'
A	2'-6½"	2'-8"	2'-10"	3'-1"	3'-4"	3'-10"

LENGTH OF PILE	WEIGHT PER FT.
20 TO 25 FT.	225 Lbs.
26 " 30 "	227 "
31 " 35 "	228
36 " 40 "	230 "
41 " 45 "	235 "

TYPICAL CONCRETE PILE TRESTLE
Slope 1½ to 1
END BENT
ANCHOR BENT
INTEMEDIATE BENT
Base of Rail
20'-0"
10'-0"
14'-0"
7'-0"
2'-6"
4'-3"
2'-3"
3'-6"
A
Pile to be painted with black band at this point to indicate proper place for attaching handling line.
⅓ LENGTH OF PILE
Hoops spaced 4" ctrs. for 4' each side of this point
⅔ LENGTH OF PILE
Only 4 Bars in nose
Hoops spaced about 8" ctrs. except as noted above
LENGTH OF PILE
8"
6"
5"
4"
3"
1'-4"

FIG. 5.—PILES AND TRESTLE BENT FOR STANDARD CONSTRUCTION.

BATTERY BOXES AND WELLS.

The high cost of metal with its tendency to corrode, and the short life and instability of timber, have created a demand for a more stable material for battery boxes, wells and chutes for the signal department of railways. The necessity for a receptacle that is waterproof, frost-proof, portable and of permanent construction, which properly protects storage and primary batteries, has been completely met by the use of the factory-made concrete product, and today this product is a standard on the railways of this country.

POLES.

With the decreasing supply of suitable material for wooden poles for telegraph and telephone service, the various railways in the country have been casting about for an adequate substitute. A number of short lines have been constructed with solid reinforced-concrete poles, within the past five years. These, however, have not proven satisfactory on account of their weight and cost. Recently, a machine has been developed by means of which a hollow concrete pole can be produced by centrifugal methods. With this machine a pole can be made of any desired length and taper, and the thickness varied to meet the load conditions. The density produced in the concrete by the pressure of this centrifugal force is remarkable. A pole thus made has sufficient strength to meet storm conditions, is not affected materially by the elements, is light in weight, permitting ease in handling, has a pleasing appearance, and is low in first cost. In addition to the requirements for telegraph and telephone service, these poles can be used on railways for station lighting and signals of all types.

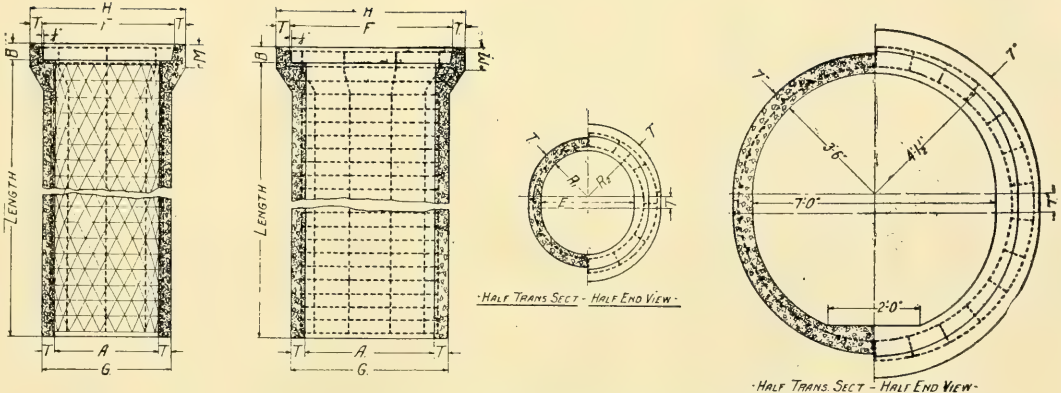
MISCELLANEOUS PRODUCTS.

In the foregoing, an attempt has been made to show the application of those concrete products in general use. Other products in concrete have been used as substitutes for timber, cast iron and steel, such as posts for right-of-way fences, hand rails for viaducts and retaining walls, pipe-carrier foundations for interlocking plants, warning signs, smoke jacks for engine houses, boot tanks for grain elevators, cribbing and crossing planks.

WHERE THE PRODUCTS ARE MADE.

The first plants for the manufacture of concrete products for railways consisted of a siding, a derrick within range of the siding, a platform around the derrick on which to set up the forms, a mixer within the radius of the derrick so that it could fill the forms, an industrial track on which was operated a car to transport the green product to the adjacent seasoning yard, a cement house, and cage shed. No attempt was made to operate these early plants during the winter, the plan being to manufacture as much product during the warm and fair weather as possible and shut down for the winter. The output of such a plant was further limited to the speed of the derrick which was used to unload raw material, set up and strip the forms, pour the concrete into them, and load the finished product.

As the demand for the factory-made product increased, it was found necessary to operate part of the winter, at least, on days when the weather was favorable. For the manufacture of pipe this was made possible by the introduction of small steam rooms within the radius of the derrick. These rooms were large enough to hold a number of pipes and had removable roofs so arranged that they could be handled easily with the derrick. It was cus-


H
F
T
B
W
M
LENGTH
A
G
E
R₁
R₂
·HALF TRANS SECT - HALF END VIEW·
7'0"
3'6"
4'1½"
2'0"
·HALF TRANS. SECT - HALF END VIEW·

E	A	G	F	H	T	M	B	R₁	R₂	WATER AREA SQUARE FEET	% EXCESS WATER AREA OVER ROUND	LENGTH OF PIPE	WEIGHT PER LIN FT POUNDS.
18"	20½"	2'1½"	2'2½"	2'7½"	2½"	6"	4"	0'9"	1'0"	208	176	8'0"	190
20"	22¾"	2'4¼"	2'5¼"	2'10¾"	2¾"	6"	4"	0'10"	1'1¼"	256	169	8'0"	233
24"	27"	2'9"	2'10"	3'4"	3"	6"	4"	1'0"	1'3½"	364	155	8'0"	300
30"	33½"	3'4½"	3'5½"	4'0½"	3½"	6"	4"	1'3"	1'7"	564	149	8'0"	435
36"	40"	4'0"	4'1"	4'9"	4"	6"	4"	1'6"	1'10½"	806	140	8'0"	590
42"	46"	4'6"	4'7"	5'3"	4"	6"	4"	1'9"	2'1½"	1071	113	8'0"	675
48"	52¼"	5'0¾"	5'1¾"	5'10¼"	4¼"	6"	4"	2'0"	2'4¾"	1397	111	8'0"	815
54"	59"	5'9"	5'10"	6'8"	5"	8"	5"	2'3"	2'8½"	1778	118	6'0"	1075
60"	65"	6'3"	6'4"	7'2"	5"	8"	5"	2'6"	2'11½"	2172	106	6'0"	1200
68"	74"	7'2"	7'3"	8'3"	6"	8"	6"	2'10"	3'4½"	2805	-004	6'0"	1700
84"	89¼"	8'9"	8'10"	10'0"	7"	8"	7"	3'6"	4'1½"	4264	10.5	6'0"	2445

FIG. 6.—STANDARD SIZES OF REINFORCED-CONCRETE RAILWAY CULVERT PIPE.

tomary to build these houses in groups of three, so that only one at a time was open each day for casting, while the other two contained pipe seasoning under steam. In the case of battery wells, boxes and houses, a separate building was erected for their manufacture. This was usually a one-story structure, with a concrete floor and some means of heating. As these products can be handled with a large horse on wheels, this building was placed beyond the range of the derrick but near enough so that the finished product when wheeled outside the building could be loaded out by the derrick.

With the further development of the industry, it became necessary to design a plant for continuous operation and large capacity. Today, a modern plant for the manufacture of concrete pipe consists of a long one-story building equipped at one end with an elevated hopper for sand and gravel, filled with a derrick and clam-shell bucket. The mixer is also elevated and is placed directly under the gates of the hopper so that the operator can control the flow of sand and gravel easily and quickly. The house is equipped with a number of parallel tracks with two transfers—one in the rear and one directly in front of the mixer. The forms are set up on platforms cars which are pushed up to the mixer, filled, and then pushed down another track to the steam room. In the middle of the house is a stripping tower equipped with electric hoists and chain blocks for stripping and setting up the forms.

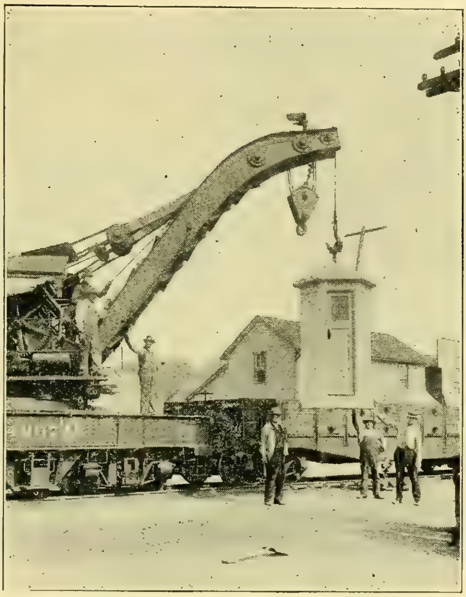


FIG. 7.—LIFTING A PRECAST RAILWAY HOUSE.

In the rear of the plant is a derrick for removing the pipe from the cars to the seasoning yard. The cement storage room and the cage room are located inside the main building. Such a plant can run twenty-four hours a day throughout the year and has a large production.

In addition to the pipe house, a modern plant has a building for the manufacture of wells, boxes, manholes, houses, posts and smaller products. It also has a pile yard for the manufacture of concrete piles and slabs, which is usually a level strip of ground adjacent to a piece of straight side track. The equipment necessary for manufacture of piles and slabs consists of a mixing plant, forms—preferably of steel, foundation timbers and pallets to support the forms and a locomotive crane for setting forms, casting, and loading the seasoned piles and slabs. For the manufacture of the hollow conrete poles a separate building is required with a mixing plant and an overhead crane for handling the raw material and the finished product. From the foregoing it is seen that the present tendency in plant construction is to obtain uninterrupted production and protect the product and the labor from the elements which results in more and better product.

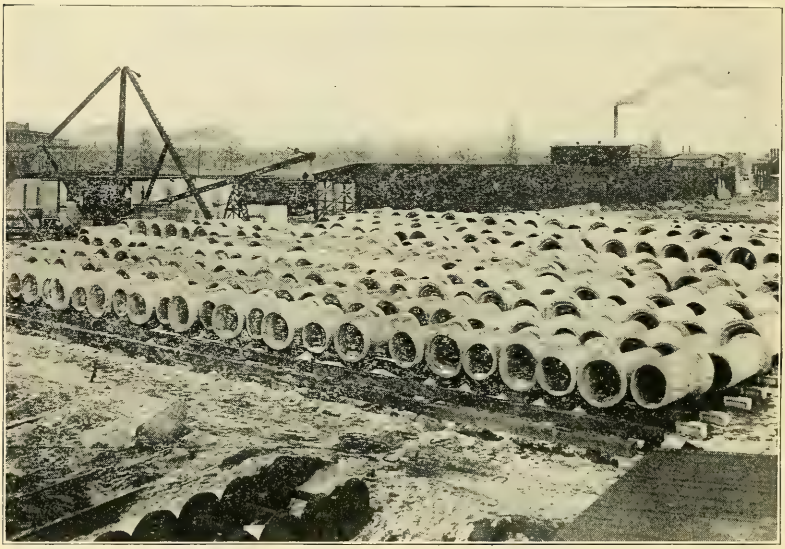


FIG. 8.—CONCRETE PIPE IN STOCK YARD AWAITING DISTRIBUTION TO CUSTOMERS.

SUMMARY.

In conclusion, the foregoing may be summed up as follows: With the use of the factory-made reinforced-concrete products, the uncertainties of other methods of construction can be eliminated and the amount of labor, equipment and transportation greatly reduced. The delays due to weather conditions, dependency on migratory labor and the necessity of maintaining traffic, regardless of construction economies, are largely eliminated. The centralized and thoroughly trained organization of the factory permits a thoroughness of inspection during all stages of production that prevents, as far as possible, a failure after installation. The work in the factory can progress continuously without regard to climatic or other conditions, making it possible to carry a stock of finished, seasoned product at all times. By taking the finished product from stock, a definite program for installation can be arranged and carried out. The thoroughness of workmanship that is possible in the factory results in a quality of concrete that can be depended upon to meet designing requirements with a maximum degree of certainty. The great density of concrete, when properly proportioned, mixed and thoroughly seasoned, such as produced under factory methods, permits the use of higher stresses than ordinarily assumed.

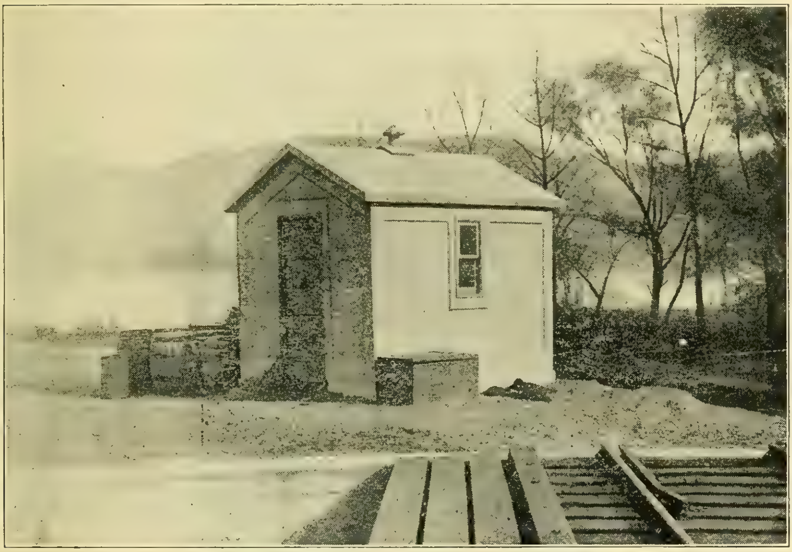


FIG. 9.—TYPE OF LARGE PRECAST HOUSE.

It is often necessary to use large factors of safety or low stresses to guard against the possibility of poor concrete in certain classes of field work, due to the prevalent use of local material, an unreliable supply of water as regards its purity and the inability to maintain thorough inspection. These conditions of course do not hold true on field work in general, but they can be eliminated more completely by the factory system than by any other. The high grade of concrete obtained by the factory methods justifies the use of a higher percentage of reinforcement with a result that sectional areas can be greatly reduced. Experience has shown that it is economically feasible to eliminate cast iron, steel, timber and other material subject to deterioration from many construction projects where formerly these materials were considered indispensable.

Today the use of the concrete product in preference to these other materials is of most importance in that the ingredients of the concrete product and the labor necessary to produce it do not interfere with the production of war materials.

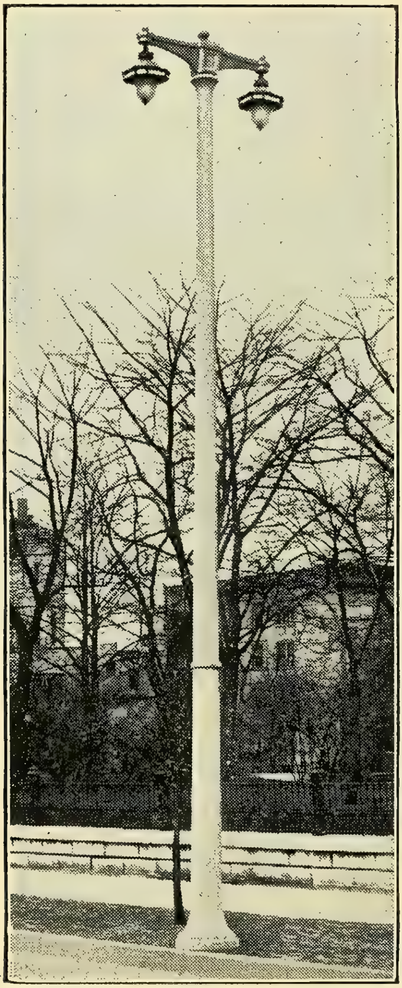


FIG. 10.—HOLLOW CONCRETE LIGHTING STANDARD.

REINFORCED-CONCRETE PRESSURE PIPE.

By Coleman Meriwether.*

In recent years precast reinforced-concrete pipe has been developed to meet the principal requirements of pressure pipe lines, such as:

Ability to resist internal and external pressures, minimum coefficient of friction, minimum leakage, minimum maintenance charges, permanency of construction, provision for contraction and expansion, and construction costs as low as is consistent to obtain the above results.

Concrete properly made and sufficiently reinforced will resist internal and external stresses and will work within safe limits up to a water pressure of 80 lb. per sq. in.

Correct methods of manufacture will produce concrete pipe with a low coefficient of friction. The Sooke Lake Water Supply Conduit for the City of Victoria, B. C., constructed of reinforced-concrete pipe 42 in. in diameter is 27½ miles long, contains 52 per cent of curves with radii varying from 90 ft. to 150 ft. and seven siphons having a maximum head of 94 ft., was tested by Wynne Meridith, of Sanderson & Porter, consulting engineers on the work, and the coefficients of friction (*n* of Kutter's formula) found to be 0.01058 at the inlet end and 0.0117 at the outlet end, distant 27½ miles, with the pipe running full at the inlet and $\frac{6}{7}$ full, or 36 in. in depth, at the outlet, readings being taken simultaneously. With 20 in. of water at the inlet the water level at outlet was 19½ in. Practice has therefore shown that greater flow has been found in lines constructed of reinforced-concrete pipe than was anticipated.

Leakage Can Be Kept Very Low.

Leakage through the walls of precast pipe has been almost nil and the leakage through the joints has been less than is usually allowable for water lines.

On a reinforced-concrete pressure pipe line constructed for the City of Baltimore several years ago, consisting of 5000 ft. of 108-in. diameter pipe and 3000 ft. of 84-in. diameter pipe tested to a head of 85 ft., the leakage in 24 hr. on the entire line amounted to 13,000 U. S. gals. or 0.062 gal. of water per sq. ft. of concrete per 24 hr. This line carries 120,000,000 gal. of water daily, and is a part of the Gunpowder Water Supply, and supplies the entire City of Baltimore with water.

Concrete is permanent, grows stronger with age and is not subject to corrosion, tuberculosis and other ills to which other materials are subject. Being of permanent construction the maintenance charges are low, and as the greater bulk of the materials are of local origin the first cost is usually lower than that of construction with other materials.

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* Manager, Cement Products Bureau, Portland Cement Association, Chicago.

Contraction and expansion will occur in pipe of any material and suitable expansion joints must be provided in concrete pressure conduits if the leakage at joints is to be kept at a minimum. Such joints have been developed for use in precast reinforced-concrete pipe and have been successful in practice. As the construction of pipe lines is usually done at temperatures higher than that of the water which will flow through the conduits, it necessarily follows that contraction will occur. This will produce cracks at the joints through which leakage of considerable amount will occur if provision has not been made to care for the contraction.

We can assume that the maximum temperature at the time of construction will be near 100° F. and that during the winter in some parts of North America the temperature will drop as low as fifty or more degrees below zero. Pipe lines should be covered with the earth backfill before work is suspended for the winter and all openings in the pipe line should be tightly closed to prevent a circulation of air in the conduit. With such precautions the temperature of the pipe line is not likely to drop below 40° F. Flowing water will have a temperature of very little under 32° F. The above assumption will give an idea of the range of temperature and the consequent contraction and expansion in water conduits.

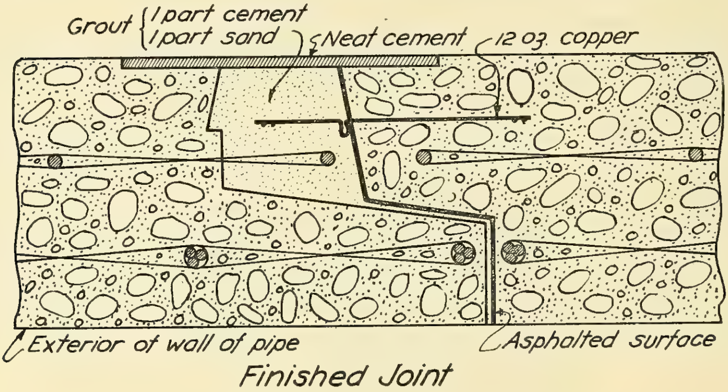

FIG. 1.—DETAIL OF WATER-TIGHT CONCRETE PIPE JOINT.

Special Joint is Water-tight.

One of the joints mentioned above (see Fig. 1) is constructed with a crimped copper band which is continuous throughout the circumference of the joint. As the pipe contracts the crimp opens and as the pipe expands the crimp closes. This joint is used in pipes of 36 in. to 108 in. in diameter and is a true expansion joint, having been found successful in different parts of North America. To reduce the number of joints it is well to make the pipes as long as is practicable and trench conditions such as bracing, etc., may be limiting factors. The practice so far has been to make precast units of a maximum laying length of 8 ft. It has also been determined in practice that it is necessary to equip each precast unit with expansion joints.

The installation of a plant for manufacturing reinforced-concrete pressure pipe for ten miles of the Greater Winnipeg Water conduit in Manitoba, Can., and some of the details of manufacture are described below, the pipe being 66 in. inside diameter of the copper expansion-joint type. The layout of such a plant is a matter of no small importance, for incessant care must be used in all the details from the installation of the plant and its equipment until the last pipe is laid and the line tested.

WINNIPEG PIPE NOTABLE INSTALLATION.

Pipe are cast on end and the molds of sheet steel and cast iron must be erected on substantial bases or foundations of reinforced concrete, the surface of the foundations being truly level and finished smoothly so that when the cast-iron base mold is set and the sheet-steel casings are erected the casings will be truly vertical. Fig 2 shows the concrete bases on the left of the derrick track and the cast-iron bases on the right of the track ready for the sheet-steel casings. In the background can be seen the forms completely assembled and being filled with concrete.

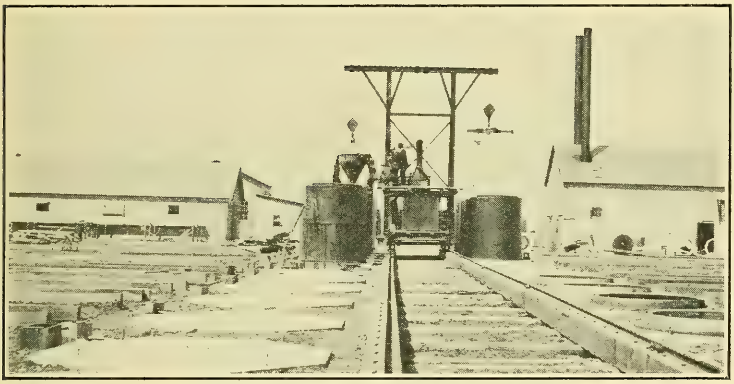

FIG. 2.—CASTING YARD FOR WINNIPEG PIPE. CONCRETE BASES ON LEFT CAST-IRON BASES ON RIGHT.

The process of assembling consists of cleaning, oiling and setting the cast-iron bases, on centers, on the concerte foundations, next placing the inner sheet-steel casings on the top of which is placed a steel filling platform which is used to center and hold truly circular the inner casings as well as to receive concrete from the conveying buckets. The inner cage of reinforcement is then placed (Fig. 3), the lower strand of which is set in an annular slot in the cast-iron base, the slot then being filled with dry sand which forms a core and prevents concrete flowing into the space and thereby binding the base to the pipe. The outer cage of reinforcement is then set and the outer steel casing placed and clamped. The mold is then ready to receive concrete.

A batch of neat cement grout is dumped on the filling platform and flows into the mold splashing over the two rings of reinforcement in its descent to the bottom of the mold where a portion of the grout remains until the first batch of concrete is deposited when this surplus grout rises with and is incorporated into the concrete and thereby replaces any grout which may have been abstracted from the concrete by the reinforcement. This is necessary to secure a pipe dense throughout its mass. The first batch of concrete is dumped on the filling platform almost as soon as the last of the grout flows into the mold and each mold is filled very quickly, no time elapsing sufficient to cause any line of separation between batches. The filling is continuous from start to finish. All parts of the molds are cleaned and oiled between each filling.

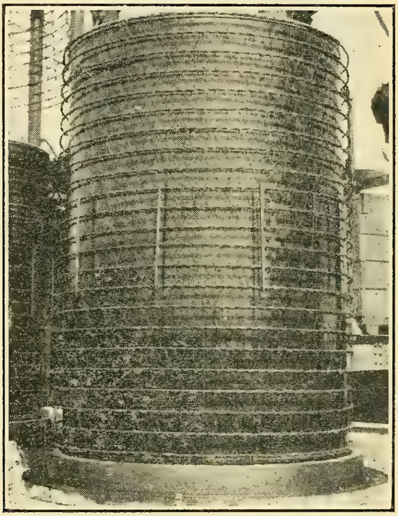

FIG. 3.—INSIDE MOLD AND PIPE REINFORCING.

MIX AND CONSISTENCY.

In the manufacture of most precast concrete pressure pipe it is necessary to use 1 volume of portland cement, 1½ volumes of sand and 2½ volumes of coarse aggregate, and in practice this means that 2½ bbl. or 950 lb. of cement is used per cubic yard of concrete. In the manufacture of precast pipe for the Winnipeg Aqueduct it was found necessary to use but one sack of cement to 3.8 c. ft. of mixed aggregate, or approximately 2 bbl. or 800 lb. of cement per cubic yard of concrete. (The Canadian barrel weighs 350 lb. gross or 346 lb. net.) This minimum quantity of cement was found practicable owing to the very excellent grading of the mixed aggregate which was supplied by the Greater Winnipeg Water District from their own pit at which was located a screening and remixing plant. The concrete is mixed to a quaking or jelly-like consistency, which will easily flow to place when slightly puddled.

The mortar for spigots on this contract is made of 1 part cement to 2 parts sand and is mixed to the same consistency as the concrete so as to obtain the same rate of setting and settlement as nearly as possible. As the spigot mortar settles more mortar is added until the settlement ceases, when the joint is finished. On the contract represented in Figs. 2–8 the concrete was mixed in batch mixers at each end of the manufacturing plant, and after being dumped into conical buckets was transported by traveling derricks to the molds. A simple ball valve easily and accurately controlled the flow of concrete from the buckets which carried grout, concrete and mortar equally well. Molds were filled simultaneously in pairs, one member of each pair being on opposite sides of the derrick track, which was located in the center of the manufacturing site. When the concrete in the molds reached the top of the outer casing a cast-iron spigot ring was set and the spigot mortar deposited, tamped in place and the copper expansion joint set. Each expansion joint is mounted on a steel spider which remains in position until the concrete has set and the forms are to be stripped from the pipe. This spider can be seen in Fig. 4 lying flat on the filling platform. The spider is centered accurately by means of a pin and a hole in the center of the filling platform.

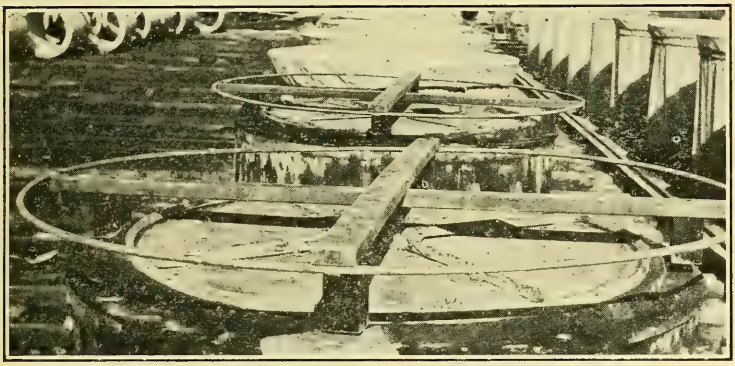

FIG. 4.—SPIDER MOUNTING FOR EXPANSION JOINT.

CURED WITH MOIST STEAM.

To maintain a constant and uniform rate of setting in the concrete as well as to protect the concrete from the low night temperatures sometimes attained in Canada, wet steam was used and a moist heat secured. The steam mains were laid underground in a way to prevent excessive radiation as well as to place the mains out of the way. A riser connected each steaming set of five vertical jets with the mains. One of the jets was placed in the center of the concrete foundation so as to be in the center of the cast-iron base and the other four jets were spaced equi-distant around the circumference

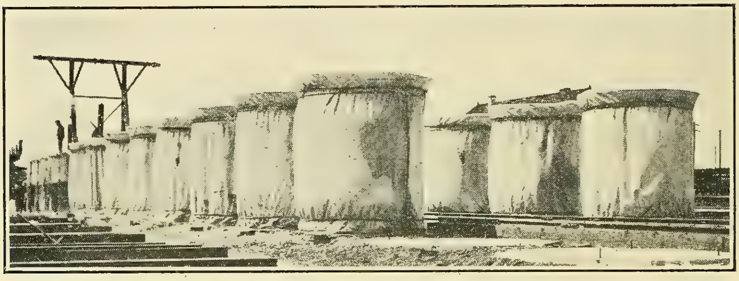

FIG. 5.—COVERINGS TO PROTECT PIPE IN COLD WEATHER.

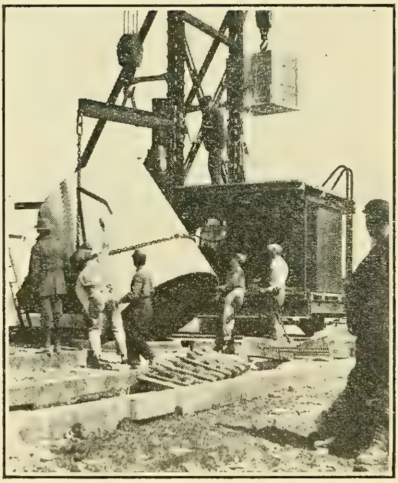

FIG. 6.—SHIFTING A 9-TON PIPE WITH TRAVELING DERRICK.

of the base, sufficient space being allowed for clearance between the jets and the edge of the base (Fig. 5). To confine the steam to the pipes canvas jackets and hoods were used, the jackets being suspended by hooks from an iron pipe ring supported on two hardwood arms which were raised above the copper expansion joint strip by hardwood blocks on each end of the arms. The jackets were laced vertically and a canvas hood placed over the top of

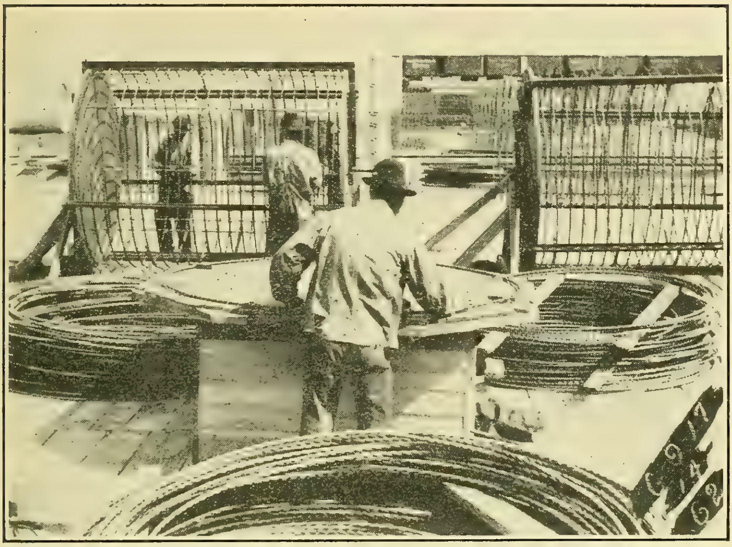

FIG. 7.—WINDING THE REINFORCEMENT FOR CONCRETE PIPE.

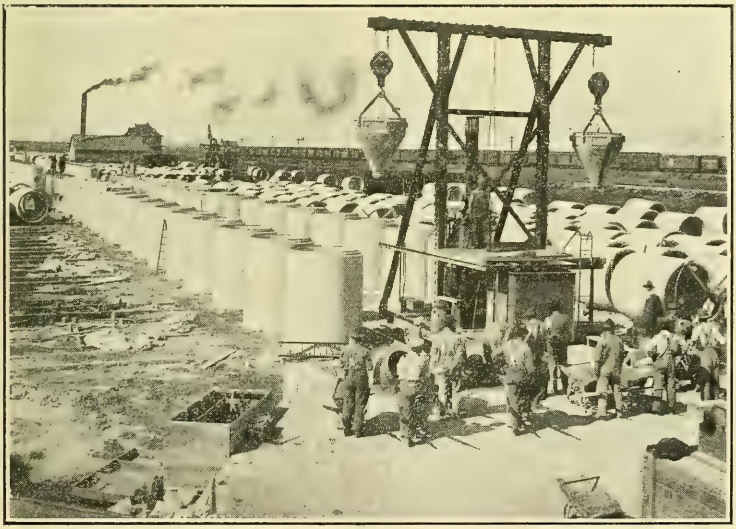

FIG. 8.—CASTING YARD FOR THE WINNIPEG PIPE.

the steam cover spider just described, the wood arms and the iron ring of the spider preventing the hood from resting on the light copper expansion joint.

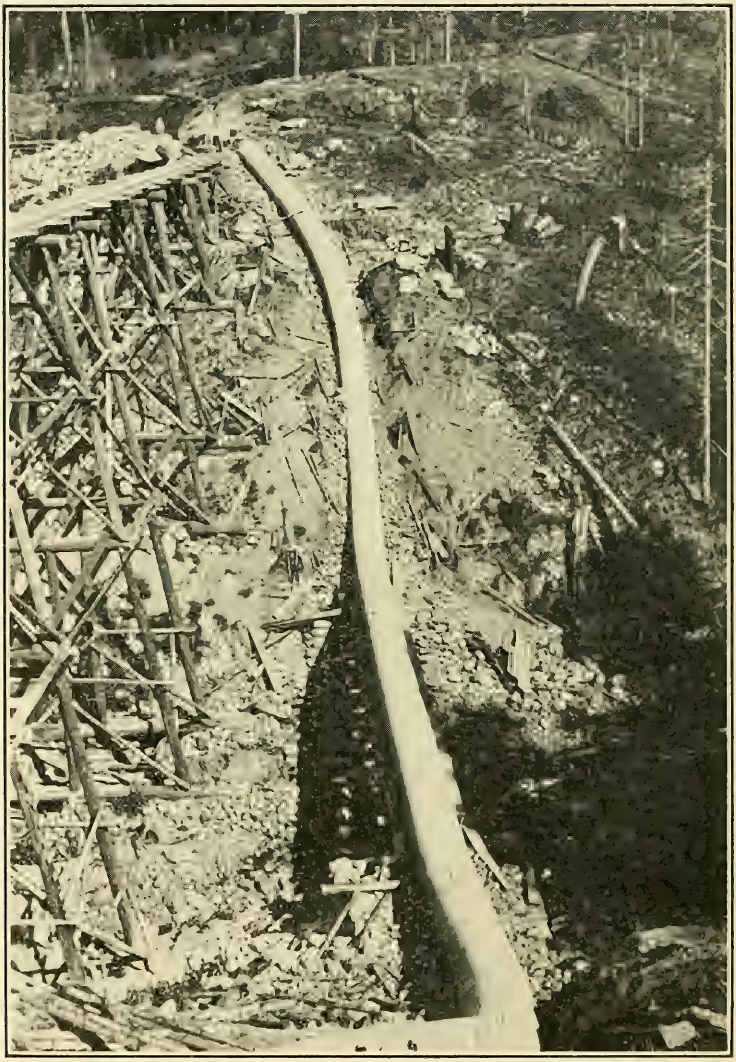

FIG. 9.—SOOKE LAKE 42-IN. REINFORCED-CONCRETE PRESSURE PIPE.

The hood was tied to the jacket by means of rope and rings, and the jackets were held snugly to place by means of rope run through eyebolts secured in the concrete bases. Fig. 4 shows the steam cover spider and Fig. 5 the jackets and hoods in place. The canvas steam covers are removed with the spider when the forms are to be stripped and are replaced as soon as the casings have been removed. The steam is again turned on and the pipe are steamed for from three to four days.

Fig. 6 shows the traveling derrick in the act of turning a pipe weighing about 9 tons, the pipe being 66 in. inside diameter, 8 ft. long and having walls 10 in. thick. After being lifted and turned the pipe are placed on heavy skids which lead to the loading tracks, the pipe being moved each day as required. Loading is done with a locomotive crane or car derrick. The pipe are being transported to the trench on flat cars drawn by a gasoline locomotive.

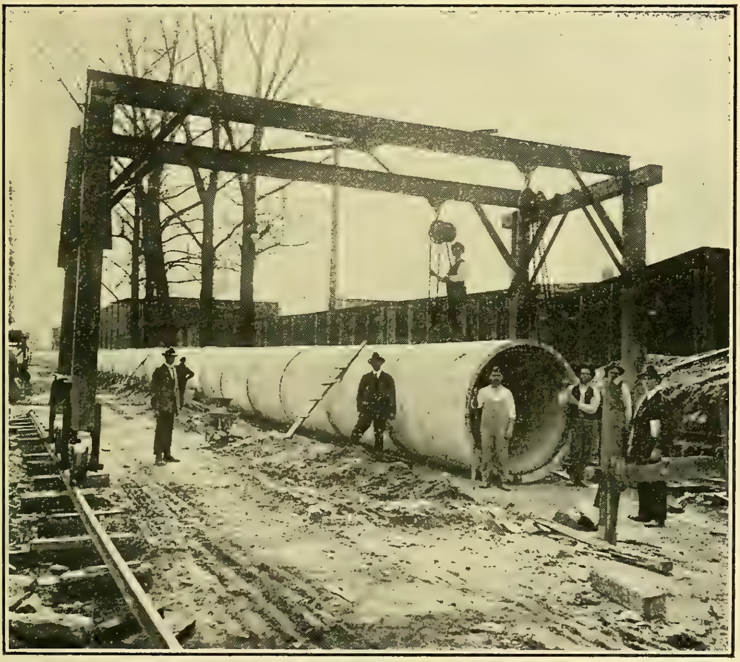

FIG. 10.—INVERTED SIPHON OF 66-IN. DIAMETER FOR FRANKFORT CREEK INTERCEPTING SEWER, PHILADELPHIA.

Fig. 7 shows a part of the work platforms on which the reinforcement is rolled and fabricated. The bars come in required lengths, are rolled to a circle, placed on a tying table, tō be seen in the foreground, tied to an accurate gage, placed on the stock piles and then taken to the squirrel cage mandrels for fabrication into cages where every bar is sure to be placed in its proper position. The mandrels are in the near background. Fig. 8 is a good general view of the plant, only half of which can be plainly seen the other half being an exact duplicate.

One of these pipe was put under a test pressure of 50 lb. per sq. in. with a leakage of 335 Imp. gal. per mile per 24 hours. This result was secured by the use of well-designed equipment, well-graded aggregate, proper methods of manufacture and unremitting care. As stated above the aggregate was almost perfect in its grading.

SOME OTHER EXAMPLES.

Illustrating what has been done on other contracts, Fig. 9 shows a view of a 42-in. reinforced-concrete pipe line 27½ miles long, built to carry water from Sooke Lake to Humpback Reservoir for the City of Victoria, B. C. The greater part of the line was under gravity head only but the various siphons, seven in number, are subjected to heads ranging from 40 ft. to 94 ft. Fig. 9 shows a siphon under 94-ft. head. The gravity portion of the line was but 3 in. thick but the siphons were constructed of pipe 4½ in. thick.

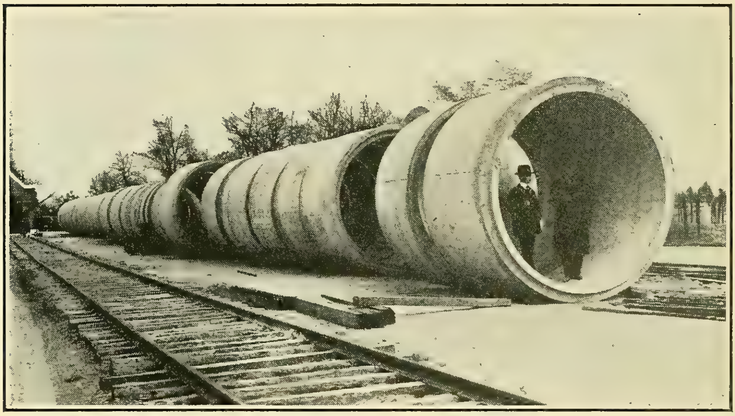

FIG. 11.—108-IN. PRESSURE PIPE FOR GUNPOWDER CONDUIT, BALTIMORE.

Fig. 10 shows a 66-in. inverted siphon in the Frankford Creek intercepting sewer for the City of Philadelphia, under a 30-ft. head.

Fig. 11 is a view of pipe for the Gunpowder Conduit, City of Baltimore, where 6000 ft. of pipe 108 and 84 in. in diameter was laid in a tunnel and 2700 ft. of 108 and 85-in. pipe was laid in open trench.

Fig. 12 shows 42-in. pipe for a water line for the City of Seattle, Wash., whch is to operate under a 90-ft. head. The pipe line in Seattle was a trunk water conduit to operate under a head of 90 ft. and every pipe was tested to 1½ times the working head. Tests were made on short sections of completed line and the leakage was nil.

FLEXIBLE TIGHT JOINT DEVISED.

All that has been said heretofore refers particularly to the copper expansion-joint type of reinforced-concrete pressure pipe. There has recently been developed a new type of expansion joint whch is indeed very efficient. An article on the subject of concrete pressure pipe would not be complete without a description of the joint and some of the principal features of the design. Reinforced-concrete pressure pipe 36 in. in diameter with this type of joint has been manufactured and under test was found to be as satisfactory as the joints for smaller size pipe.

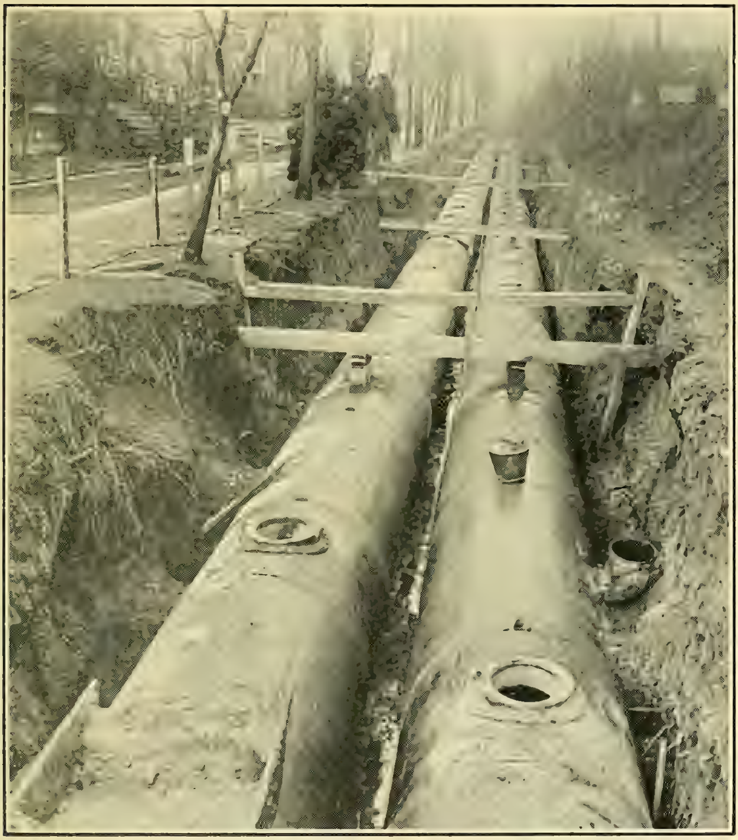

FIG. 12.—TRUNK WATER CONDUIT AT SEATTLE, WASH. TO BE UNDER HEAD OF 90 FT.

It is proposed to construct reinforced-concrete pressure pipe in diameters of 10 to 48 in. and in lengths of 12 ft., each section of pipe being provided with a cast-iron spigot ring at one end and a cast-iron bell ring at the other the rings being cast integrally with the concrete. Fig. 13 shows the details of the joint in different stages of connecting the pipe.

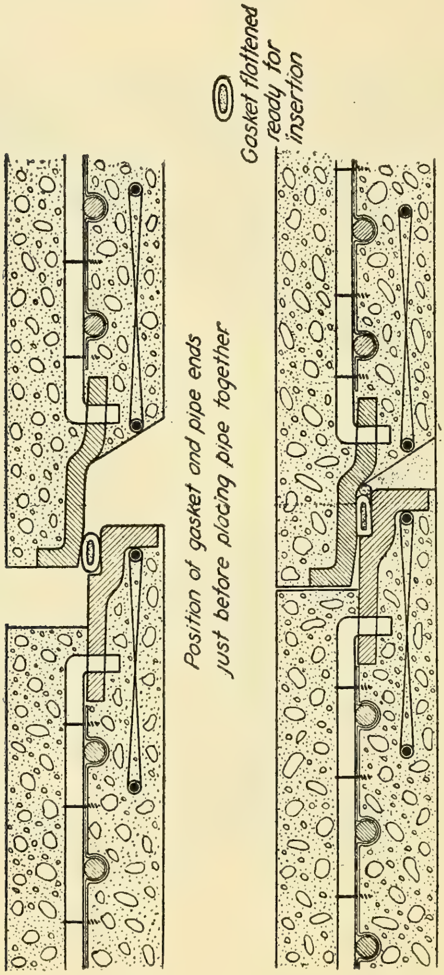
Gasket flattened ready for insertion
Position of gasket and pipe ends just before placing pipe together
Finished Joint

FIG. 13.—DETAILS OF A NEW SPECIAL WATER-TIGHT FLEXIBLE JOINT FOR REINFORCED-CONCRETE PIPE.

The faces of the rings which bear upon the lead gasket will be accurately machined providing a very true circular surface. The spigot ring is provided with a seat for the lead gasket, the face of which seat forms half of a dovetail, the object being to provide a greater thickness of gasket at the seat of the dovetail. This prevents the gasket being withdrawn when the pipe contracts or when the pipe is deflected.

The lead gasket consists of a lead pipe filled with compressed fiber and then flatten to and elliptical section, the proper length of gasket being turned to a circle which is joined forming a ring. This ring is placed in the bell and the pipe is then ready to receive the spigot end of the next pipe to be laid. As the pipe are shoved home the lead gasket is changed from the elliptical section to a section which fills the dovetail space and the space between the dovetail of the bell and the outer face of the spigot. After this has been done a light rope of cotton or jute is placed and a weak joint filler of cement mortar is applied filling the calking space, which calking space is provided in the event it should be necessary to calk the lead gasket joint. Such calking is not expected to be required but is to provide for every possible contingency.

This pipe can be manufactured at or near the site of its final location and some of the details of manufacture would be different from the processes employed in the manufacture of the copper expansion-joint reinforced-concrete pipe. Unlike the type of joint ordinarily used, this joint is tight under pressure when expanded, on account of the pipe contracting or being deflected. It has been tight under test at 110 lb. pressure.

DESIGN OF REINFORCED-CONCRETE CHIMNEYS.

By J. G. Mingle.*

Independent chimney structures were first used in several European countries about the last of the nineteenth century. The early chimneys were built of common brick laid in mortar. Chimney theory at that time was very meagre and all of the structures were designed by rule of thumb methods, little attention being paid to an economical design or to any architectural beauty. Most of these chimneys were of comparatively low height, usually not over 100 ft., and were built square in section. They were liberally designed and exceptionally well built as is attested to by the fact that several of these structures are still standing and appear as good as new.

Chimneys were first built in the United States during the early forties. They too were of common brick, liberally designed and well built. Shortly after the development of the steel plate industry short-guyed steel chimneys, or stacks, were built, and later on high self-supporting steel stacks came into use. As the plate industry grew and the material became cheaper, increasing numbers of them were built and they displaced the common brick variety almost entirely. Steel chimneys however did not last long and this led to the development of the perforated radial brick chimney which could be built about as cheaply as a steel chimney and yet was more permanent.

With the beginning of the twentieth century, the use of reinforced concrete as a material of construction came rapidly into prominence. This led to the development of the reinforced-concrete chimney. The first chimneys of this type were more or less of an experiment and it was some time before the engineering profession would place any confidence in them. It was found however that they could compare very favorably with the radial brick and steel types in price and, moreover, were built of materials which could be more easily assembled. After several were built increased confidence was shown in them and their growth became rapid. According to Thompson the first reinforced-concrete chimney was constructed in 1898 for the Pacific Coast Borax Co., at Bayonne, N. J., by the Ransome and Smith Co.

Features of a Concrete Chimney.

Reinforced concrete is an excellent material for a chimney. Concrete and steel have practically the same coefficient of expansion, that of concrete being 0.0000055 and of steel 0.0000065. With any degree of heat then the two materials will expand and contract alike. Hence there can be no separation of materials due to a change in temperature. Since plain concrete is very inelastic and incapable of resisting high tensile stresses, steel rods are imbedded within the walls to resist these stresses when they occur. Concrete

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* Engineer, The Rust Engineering Co., Pittsburgh, Pa., and Cleveland, Ohio.

also cannot withstand the long-continued action of an intense heat. As a protection from such action a lining of a more refractory material is built inside the chimney.

The essential features of a reinforced-concrete chimney are the shaft or outer shell, the lining or inner shell and the footing or foundation. The shaft is that part which is above the ground and exposed to the action of the wind. The lining is built independent of the shaft, an air space of from 2 to 4 in. separating the two, and usually rests on top of the footing. The footing is the reinforced-concrete slab upon which the shaft and the lining rest, the former being integrally connected to it. The footing insures the stability and safety of the entire structure. The intimate connection between the shaft and the foundation gives what is called a monolithic structure.

Reinforced-concrete chimneys may be classified into three general types according to the manner of construction; straight cylindrical, offset cylindrical, and tapered.

The first two types are now seldom built and need not be discussed here. They represent the logical development from the earliest type built to the present tapered type.

Practically all of the reinforced-concrete chimneys built today are of the tapered type. As suggested by its name, the outside diameter and the wall thickness of this type of chimney taper or increase from the top downwards. The percentage of taper and wall thickness is determined by formula and experiment and is governed by the height and the diameter of the chimney.

There are two systems of reinforcement within the shaft, the vertical reinforcement and the horizontal reinforcement. The vertical reinforcement consists of either plain round, twisted square, or deformed steel rods and resists the vertical tension due to wind pressure, temperature, earthquake, and eccentricity. The horizontal reinforcement consists of small diameter rods spaced closely or of a fabricated mesh, and resists the horizontal tension due to temperature, and diagonal tension. The reinforcement is placed from 2 to 3 in. from the outside surface of the concrete. The horizontal reinforcement is placed on the outside of the vertical rods and is tied to it at several points.

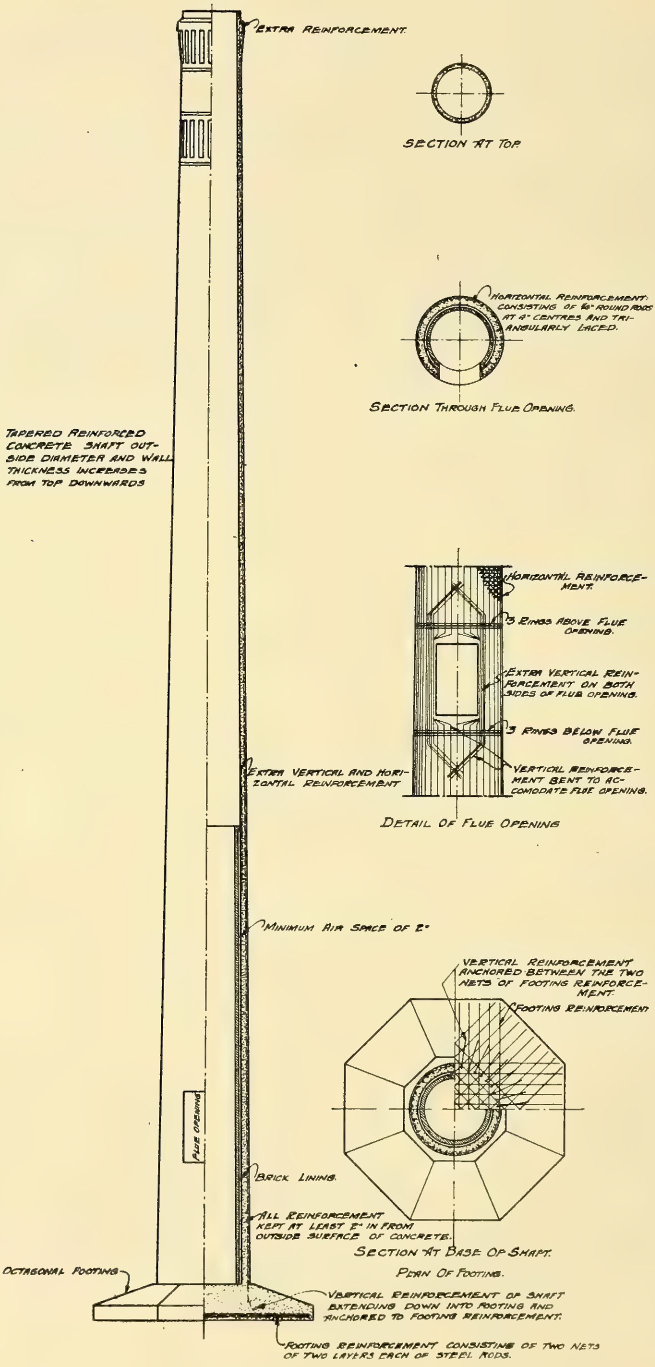
The material and height of the lining of a reinforced-concrete chimney depend upon the degree of heat to which the chimney may be subjected.

The purpose of the footing or foundation is to provide means of stability to the entire structure. The footing is reinforced at the bottom to resist the tension produced by the upward pressure of the soil. The vertical reinforcement from the shaft extends down into the footing and is anchored to the footing reinforcement.

Reinforced-concrete chimneys are used in connection with boilers, coke ovens, stills, furnaces, smelters, and other purposes.

The accompanying figure shows the general outline together with the details and the essential features of a tapered reinforced-concrete chimney used in connection with boilers.

An attempt will be made in the following paragraphs to outline a few of the essential features of the design of a tapered reinforced-concrete chimney.


EXTRA REINFORCEMENT.
TAPERED REINFORCED CONCRETE SHAFT OUTSIDE DIAMETER AND WALL THICKNESS INCREASES FROM TOP DOWNWARDS
EXTRA VERTICAL AND HORIZONTAL REINFORCEMENT
MINIMUM AIR SPACE OF 2"
FLUE OPENING
BRICK LINING.
ALL REINFORCEMENT KEPT AT LEAST 2" IN FROM OUTSIDE SURFACE OF CONCRETE.
OCTAGONAL FOOTING
VERTICAL REINFORCEMENT OF SHAFT EXTENDING DOWN INTO FOOTING AND ANCHORED TO FOOTING REINFORCEMENT.
FOOTING REINFORCEMENT CONSISTING OF TWO NETS OF TWO LAYERS EACH OF STEEL RODS.
HORIZONTAL REINFORCEMENT: CONSISTING OF ⅝" ROUND RODS AT 4" CENTRES AND TRIANGULARLY LACED.
HORIZONTAL REINFORCEMENT.
3 RINGS ABOVE FLUE OPENING.
EXTRA VERTICAL REINFORCEMENT ON BOTH SIDES OF FLUE OPENING.
3 RINGS BELOW FLUE OPENING.
VERTICAL REINFORCEMENT BENT TO ACCOMODATE FLUE OPENING.
VERTICAL REINFORCEMENT ANCHORED BETWEEN THE TWO NETS OF FOOTING REINFORCEMENT.
FOOTING REINFORCEMENT

SECTION AT TOP.

SECTION THROUGH FLUE OPENING.

DETAIL OF FLUE OPENING

SECTION AT BASE OF SHAFT.

PLAN OF FOOTING.

In order to discuss intelligently the stability of a chimney it must first be determined in what manner it might fail structurally. Considering the entire chimney as a free body the manner of failure may be as follows: by crushing of the concrete at the leeward edge of the footing, by tension between the bottom of the footing and the soil underneath at the windward edge, or by overturning about the leeward edge of the footing. Considering the shaft alone as a free body the manner of failure may be as follows: by crushing of the concrete at the leeward side, by tension in the concrete at the windward side, or by overturning about leeward edge. In any case then a chimney structure may fail by compression, tension or overturning.

Design of Shaft.

In the design of the shaft the following primary assumptions are made: invariability of plane sections, compression borne entirely by concrete, and tension resisted entirely by steel.

Considering the shaft of the chimney it is evident that the weight, and consequently the compressive stress, increases from the top downwards. Any compressive stress greater than that of a predetermined value must be taken care of by an increase in sectional area. This very increase in sectional area adds to the growth of the weight, and the sectional area immediately below must be further increased as a consequence. The sectional area at any horizontal section through a shaft of any height may be found from the formula

$$\log a = \log \frac{W}{f_c} + 0.4343 \frac{wh}{f_c} \quad \text{.....................} (1)$$

where a = sectional area in sq. ft.,
W = superincumbent weight in lb.,
f_c = maximum compressive stress in lb. per sq. in.,
w = weight per cu. ft. of material,
h = length of shaft in ft.

Assuming a value of 350 lb. per sq. in. for f_c and 150 lb. per cu. ft., for the w, the formula reduces to

$$\log a = \frac{\log W + 65.15h}{350} \quad \text{.....................} (2)$$

The above formula is very flexible and permits of a wide variation of diameter and wall thickness increments. It is limiting for compression only and has nothing to do with the tension. Only by the cut and try method can a combination of wall thickness and diameter increments be found which will be theoretically correct, and economically the cheapest. Also practical considerations and the comparatively unknown effect of the heat should be taken into account. Enough taper should be provided to give the chimney a graceful outline and the wall thickness should be made as thin as practicable in order to keep as low as possible the stress due to heat. At any rate the sectional area at any section is made just large enough that the maximum compressive stress in the concrete does not exceed safe values. The tension then, if any, is resisted by steel reinforcement. Outside diameter increments vary from $\frac{1}{4}$ in. to $\frac{3}{8}$ in. increase per foot and wall thickness increments from $\frac{1}{16}$ in. to $\frac{1}{32}$ in. per foot.

After having adjusted the diameter and wall thickness increments the amount of vertical and horizontal reinforcement must be determined. The purpose of the vertical reinforcement is to resist the tension due to wind pressure, temperature, earthquake and eccentricity, and of the horizontal reinforcement, to resist the tension due to temperature and diagonal tension.

WIND PRESSURE REINFORCEMENT.

When a chimney is under strictly static conditions, that is, when there is no wind blowing aginst it, the stress on any plane horizontal section is entirely compressive and uniformly distributed. When wind blows against the shaft the distribution of the stress is altered, that at the windward side being decreased and at the leeward side increased. When the resultant migrates outside the kern, tension results at the windward side. This tension must be resisted by steel reinforcement, the amount of which at any section may be determined from the formulas

$$A = \frac{T}{f_s} A_c \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

or

$$A_s = \frac{2M}{f_s R_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where A_s = total amount of reinforcement in sq. in.,
f_s = steel stress in lb. per sq. in.,
A_c = sectional area in sq. in.,
M = bending moment in in. lb.,
R_s = radius of reinforcement centers in inches.

The tension is equal to the stress due to wind less the stress due to weight. The bending moment is equal to the moment due to wind pressure less the moment due to weight or the statical moment.

Steel percentages for wind pressure vary from 0.0025 per cent to 0.03 per cent, or greater.

Most chimneys are designed to withstand a wind blowing with a velocity of 100 miles per hour. The unit wind pressure to be assumed is a mooted question. The present general practice assumes a unit pressure of 25 lb. per sq. ft. on the diametral or projected area, although many building ordinances recommend a unit of 20 lb. per sq. ft. Due to lack of test data of wind pressure on cylindrical surfaces, the question will remain an open one until more such tests have been made.

TEMPERATURE REINFORCEMENT.

When a reinforced-concrete chimney is in use the inside of the shaft is hotter than the outside, consequently the inside tends to expand more vertically and circumferentially than the outside. At the same time tensile stresses occur in the outermost fibers. When the tension in the outermost fibers becomes excessive, vertical and horizontal cracks may appear. These cracks frequently appear as fine hair line cracks and are not serious unless they open up. To prevent these cracks from starting, sufficient vertical and circumferential reinforcement must be provided to resist these stresses.

For ordinary boiler temperature up to say 500° F., the vertical tension due to temperature is perhaps slight. But for temperatures above this figure and especially above 1,000° F., the stress is of some consequence and should be taken into account. There is no practical formula for determining the vertical tension due to temperature. Several authors have developed a formula which indicates in a way an approximation, and from this have determined the percentage of reinforcement. The percentage of reinforcement is dependent upon the difference of temperature and the thickness of the wall.

It is evident that in order to keep the stresses due to temperature down to a minimum the wall should be made as thin as possible.

EARTHQUAKE REINFORCEMENT.

When reinforced-concrete chimneys are built in regions subject to earthquake it becomes necessary to provide additional reinforcement at a certain region in the shaft to resist the tensile stress caused by this disturbance. The maximum stresses due to earthquakes do not occur at the base of the shaft, as is the case of wind pressure, but at a height equal approximately to $\frac{2}{3}H$ which corresponds roughly to the center of percussion considering the chimney as an inverted pendulum.

The force due to earthquakes is equal to

$$P = k\frac{w}{g} \qquad (5)$$

where k = acceleration due to earthquake ft. per sec. per sec.,
w = superincumbent weight in lb.,
g = acceleration due to gravity ft. per sec. per sec.

The acceleration due to earthquake is taken at 7 to 9 ft. per sec. per sec. The force due to earthquake is considered as applied instantaneously.

The amount of reinforcement necessary to resist this stress may be computed from formulas 3 and 4. This reinforcement is placed in the region of $\frac{2}{3}H$ from the base.

CIRCUMFERENTIAL REINFORCEMENT.

It is impossible to develop an exact formula for determining the circumferential stress due to temperature. Temperatures vary between wide limits, and it is hardly possible to make an accurate assumption of the difference of temperature between the inside and outside surfaces of the concrete. Also it is not known definitely just how concrete acts within the range of variation.

The circumferential reinforcement resists the circumferential stress due temperature or rather distributes the stress so as to prevent the localization of cracks. It is apparent that the reinforcement should be spaced as closely as possible and should be well distributed. A good ready-made fabrication on the market for circumferential reinforcement is the American Steel and Wire Co. Triangular Mesh Concrete Reinforcement. A similar material may be used. It is made up of small diameter rods spaced about 4 in. center to center and triangularly laced with a small gauge wire. This fabrication has an ideal arrangement and moreover is easily handled and placed in position to be concreted in the shaft. The mesh is placed in the outside of the vertical reinforcement and is tied to the latter at several points.

Theoretically the percentage of circumferential reinforcement varies as the difference in temperature, being greater in the vicinity of the flue opening than at the top of the shaft. Practically, however, the same amount is placed at the different sections throughout the entire shaft.

The region of the shaft at the top of the lining should be doubly reinforced with the mesh. Intense internal stresses occur at this point due to a sudden jump from a partially cooled shaft to a greatly heated interior surface. It is also well to add a few more vertical rods in this region to resist the increased vertical tension.

The external forces which act in the shaft of a reinforced-concrete chimney produce, in addition to vertical tension, which is resisted by vertical reinforcement, shear or diagonal tension, must be resisted by circumferential reinforcement. The stress due to diagonal tension is a function of the vertical tension and is usually not very great. It is usually taken for granted that in average size chimneys the circumferential reinforcement necessary to resist the temperature stress will also take care of the diagonal tension. In extreme cases however, that is in cases where the height of the chimney is very great in comparison with the greatest wall thickness, additional reinforcement may be necessary. The percentage of diagonal tension reinforcement may be computed from the formula

$$p = 0.05 \frac{HP}{tf_s} \qquad (6)$$

where H = height of chimney in ft.,
p = unit wind pressure in lb.,
t = wall thickness in in.,
f_s = allowable steel stress in lb. per sq. in.

DESIGN OF FOOTING.

The footing or foundation is perhaps the most important part of the chimney, for upon it depends the stability and safety of the entire structure. A chimney may be compared to a candle with its candlestick. A candle when made to stand alone in an upright position is very unstable and will easily overturn, but when placed in a candlestick it becomes stable and a considerable force is required to overturn it. By supplanting the candle with the shaft and the candlestick with the footing of the chimney, the analogy is seen.

The footing serves a twofold purpose to the chimney:

(1) To prevent the structure from overturning when under the action of the wind,

(2) To supply a medium for distributing the weight of the entire structure on the soil underneath in such a manner that the safe bearing capacity of the soil will not be exceeded.

The above may be called conditions of safety. Both conditions should be investigated in every instance, as the footing may be large enough to prevent overturning and yet the safe bearing capacity may be exceeded or the soil pressure may be safe and at the same time there may be a tendency for the structure to overturn.

The greatest thickness of the footing depends upon the intensity of shear it is to resist and varies from 3 ft. for a 100-ft. shaft to 7 ft. for a 300-ft. shaft. Local conditions sometimes require a greater or less thickness of footing. The footing slopes from the greatest thickness at the base of the shaft to a thickness slightly less than half the greatest thickness at the edge, depending upon the length of the outstanding arm. The bottom of the footing should always be at least four feet below the surface of the ground or at any rate below the average frost line.

The size of the footing is dependent upon two conditions: the maximum local soil pressure and a no tension condition at the windward toe. The footing may be built square, octagonal or circular in section. Extremely large footings are usually built circular in shape, while average size footings are usually built octagonal. Square footings are uneconomical and are subjected to very large toe pressures when the wind blows parallel to a diagonal.

The maximum soil pressure underneath the footing at the leeward edge when the chimney is being subjected to a 100-mile-per-hour wind is found from the formula

$$P = \frac{W}{A} + \frac{Mc}{I} \quad \ldots\ldots\ldots\ldots (7)$$

and should not exceed the safe bearing capacity of the soil. The table below, taken from the American Civil Engineer's Pocketbook, gives the safe bearing capacity of different soils in tons per square foot.

	MIN.	MAX.
Rock, the hardest in thick layers in natural beds....	200	..
Rock, equal to best ashlar masonry...............	25	30
Rock, equal to best brick masonry...............	15	20
Rock, equal to poor brick masonry...............	5	10
Clay, in thick beds, always dry..................	6	8
Clay, in thick beds, moderately dry..............	4	6
Clay, soft......................................	1	2
Gravel and coarse sand, well cemented............	8	10
Sand, dry, compact and well cemented.............	4	6
Sand, clean, dry................................	2	4
Quicksand, alluvial soils, etc..................	0.5	1

Practical considerations, however, should determine the safe bearing capacity in every case. In view of the fact that only a slight settlement underneath the footing will endanger the entire structure or throw it out of plumb, the above values should be used conservatively. The maximum soil pressure of chimneys of ordinary height seldom exceeds 6,000 lb. per sq. ft., but with chimneys 300 ft. high and over the soil pressure may greatly exceed the above figure.

The bottom of every footing should have sufficient area to preclude any tendency to tension at the windward toe. Referring to formula 7, for a no tension condition at the windward toe,

$$\frac{W}{A} = \frac{Mc}{I} \quad \text{.............................(8)}$$

whence $d = \dfrac{6M}{W}$ for square footing,

$d = \dfrac{7.53M}{W}$ for circular footing,

$d = \dfrac{8M}{W}$ for circular footing,

where d = greatest face to face dimension.

REINFORCEMENT.

In determining the amount of reinforcement in the bottom of the footing, the footing is considered as a cantilever beam with the soil pressure underneath as a loading. Two separate cases will occur.

(1) A positive moment in the bottom of the cantilever arm of the footing when the length of the cantilever arm is equal to or more than one-half the inside diameter of the chimney at its base, and which will be a maximum when the chimney is being subjected to a 100-mile-per-hour wind.

(2) A negative moment in the top of the footing on the area within the inside diameter of the chimney, when one-half of the inside diameter of the shaft at the base is greater than the length of the cantilever arm, and which will be a maximum when there is no wind blowing against the chimney.

POSITIVE MOMENT IN FOOTING.

The positive moment occurs in the bottom of the outstanding arm of the footing and is a maximum when the chimney is being subjected to a 100-mile-per-hour wind. The amount of the moment is equal to

$$M_p = P\frac{d}{2} \quad \text{.............................(9)}$$

where P = maximum resultant soil pressure.

The amount of reinforcement necessary to resist this moment is equal to

$$A_s = \frac{M}{f_s jd} \quad \text{.........................} (10)$$

The reinforcement is divided into two nets of two layers of steel each, spaced about 2 or 3 in. apart vertically. The vertical reinforcement of the shaft extends down into the footing and is anchored in between these two nets. Each of the two nets is divided into two layers, thus making four different layers of reinforcement. In order to secure the best distribution each of the layers is to run in a different direction. The best arrangement is to have the two layers of the lower net at right angles to each other and the two layers of the upper net also at right angles to each other but at 45° with the lower net. The bottom net is placed about 4 in. up from the bottom of the footing and the top net from 2 to 3 in. above the bottom net.

It should be borne in mind that the steel area, as determined above, is the maximum, and that this occurs in the bottom directly underneath the base of the shaft. Theoretically, less reinforcement will be required towards the toe and underneath the center of the chimney, but under average conditions it would be impractical to vary the size and spacing of the rods in a lengthwise direction. For this reason the maximum amount is determined and this amount distributed throughout the entire footing. Under average conditions it is not necessary to extend the parallel rods beyond a line tangent to the outside diameter of the shaft at the base.

NEGATIVE MOMENT IN FOOTING.

The negative moment occurs in the top of the footing and is a maximum when there is no wind blowing. This is the case of a uniformly loaded beam. The maximum moment occurs at the center of the structure and is equal to

$$M_n = \frac{P\,D}{12} \quad \text{.........................} (11)$$

where D = inside diameter of shaft at the base.

Then, as before, $A_s = \dfrac{M}{f_s jd}$.

The amount of negative reinforcement is usually small and it is usually necessary to place only two layers of steel at right angles to each other. This reinforcement is placed about 2 in. down from the top of the footing and is in addition to that which is required in the bottom. It is only in rare instances that this case will occur and then in cases where the diameter of the chimney is large in proportion to its height.

LINING.

The purpose of the lining is to protect the concrete walls from the impinging action of any extreme heat. The lining is independent of the shaft except in rare instances, and rests directly on top of the footing. The height of the lining is determined by the degree of heat to which the chimney may be subjected and the location of the flue opening with reference to some datum plane. For ordinary temperatures up to 800° F., the lining should extend from

20 to 30 ft. above the top of the flue opening; for temperatures between 800° and 1200° from one-half to two-thirds of the height of the shaft; and above 1200° the chimney should be lined its entire height.

The materials of which the lining should be built depends upon the degree of heat to which the chimney may be subjected and also to the constituents of the flue gases. For ordinary boiler purposes where the temperature does not exceed 800° F. and the flue gases do not contain any large percentage of destructive gases, the lining may be built of common building brick. For temperatures above 800° the lining is built of either fire or radial brick; a second grade fire-brick or radial brick for temperatures between 800° and 1200°, and a first grade fire-brick for temperatures above 1200°. Brick in linings for high temperatures is laid in a fire-clay or a fire-clay mortar. Linings for smelters and stills require special consideration as do cases where destructive gases are encountered.

FLUE OPENING.

A section through the flue opening of a reinforced-concrete chimney represents the weakest section of the entire structure and special provision must be made to strengthen the chimney at this point. Practically all of the few failures of this type of chimney may be attributed to weaknesses around the flue opening.

When a flue opening is placed in the shaft considerable of the concrete of a section through the opening is left out. Consequently there is a proportionate reduction of area which results in much higher stresses due to both weight and wind. Moreover, the distribution of the horizontal and vertical reinforcement is also altered. The above remarks are more pertinent when there are two or more openings at the same level. At any rate, inherent weaknesses are induced by the presence of the opening, and special provision must be made to offset this undesirable condition.

In the case of a brick chimney, a large buttress is built around the flue opening to compensate for the reduction of area. The faces of the buttress extend for some distance beyond the face of the shaft of the chimney. This method is not used on a reinforced-concrete chimney, due to constructional difficulties and also on account of its impracticability. The reduction of area is compensated for by simply thickening the remainder of the wall on the inside to the extent that there will practically be no reduction of area whatever through the section. This thickening process should be carried out for a distance of approximately 5 ft. above and below the opening. Ordinarily with an opening one-half as wide as high, the wall thickness will be increased from 1 to 3 in. depending upon the percentage of reduction.

From the above remarks it will be seen that it is best to make the opening as narrow as possible. The best practice is to make the width equal to one-half the height.

Inasmuch as there is a decided tendency for chimneys to crack in the region of the flue opening, extreme care should be used that there is sufficient reinforcement around the opening and that it is properly placed.

All vertical rods should be bent into the sides to accommodate the opening. It is bad practice to cut the vertical rods which extend towards the opening since, when cut, they lose their continuity and perchance might be left out altogether. They should be bent into the sides with a gradual bend. Care should be taken that all bent rods be kept apart and not bunched. At least three extra vertical rods of the same size as the others at this region should be bent and placed on each side of the opening. They keep the corners of the opening from cracking on a line parallel to the diagonal. Three rings of rods of the same size as the vertical reinforcement are placed both above and below the opening. These rings resist the arch action, if any, and support the concrete above the opening especially if the opening is very wide. These rings also help to resist the enormous temperature stresses which are always prevalent around this region. The horizontal reinforcing mesh which ordinarily is placed about 2 in. from the outside surface, is continued into the sides of the opening and then entirely around the inside of the chimney not less than 2 in. from the surface. The two strips of mesh will serve to resist the temperature stresses and distribute them uniformly throughout the entire region. The extra mesh on the inside should be extended about 5 ft. above and below the opening. The mesh should be placed on the outside of all of the vertical rods, and should be tied to them at several places.

In chimneys which are subjected to high temperatures, the reveals of the flue opening should be lined with a fire brick. In this case the opening in the concrete walls should be made larger to accommodate the thickness of the brick.

CONCLUSION.

In conclusion, particular attention should be directed to the following points in the design of a tapered reinforced-concrete chimney:

Provide sufficient taper to the shaft to give a graceful outline.

Make wall thickness as thin as practicable.

Provide sufficient reinforcement to resist the tension due to wind pressure, temperature, earthquakes, eccentricity, and diagonal tension.

Properly and sufficiently reinforce the regions at the top of the lining and around the flue opening.

Provide a footing of sufficient size.

Properly reinforce the footing.

Compensate for the presence of the flue opening.

Make lining of sufficient height.

Use proper materials in the lining.

Use unit stresses commonly accepted in engineering practice.

Pay particular attention to soil conditions.

Use a proper concrete mix.

Do not use the chimney too soon after completion.

Careful consideration of the above points together with the added precaution of careful and intelligent construction will give a structure which will be everlasting.

DISCUSSION.

Mr. Godfrey. MR. EDWARD GODFREY (*by letter*).—This paper contains a number of excellent features. It is significant that Mr. Mingle assumes that the concrete alone takes the compressive stress due to the weight of the chimney. He does not introduce the alleged aid supplied by the steel rods. I was severely criticized by Prof. E. R. Maurer, in *Engineering News*, Oct. 1, 1908, for ignoring the compressive stress in the vertical rods.

It is also significant that Mr. Mingle uses a unit stress in compression of 350 lb. per sq. in. on the concrete. In a report on reinforced-concrete chimney failures by S. E. Thompson, published in *Engineering News*, Jan. 9, 1908, Mr. Thompson recommends a unit stress of 600 lb. per sq. in. in compression on the concrete. The writer always considered this to be entirely too great and unsafe. It is on a par with the high units on plain and rodded columns that gradually being discarded. It is to be hoped that the Committee on Reinforced-Concrete Chimneys, will recommend a unit stress of about the intensity given in Mr. Mingle's paper.

Mr. Mingle states that practically all the reinforced-concrete chimneys built today are of the tapered type and that to offset cylindrical type is seldom built. It is a good sign that this offset type has been discarded. S. E. Thompson, in the report already referred to, mentioned several chimneys that had failed at this offset and showed photographs of one that had bulged out badly and had bad vertical and horizontal cracks just below the offset. But he missed entirely the significance of these failures and their connection with this feature of design.

Mr. Mingle recommends a thickening of the wall of the chimney at, below and above the openings for breeching. This, too, is an excellent and necessary recommendation. Some chimney failures, at least of other material than concrete, have been due to weakness at openings. Still another commendable recommendation is that reinforcing rods in the footings lie under the chimney and be placed in four directions, not in the common way where two layers are used and these are spaced equally across the slab.

I believe I am justified in pointing out these evidences of progress that I anticipated in a book and in technical journals ten years ago.

Mr. Shainwald. MR. R. L. SHAINWALD.—I would like to ask Mr. Mingle's advice as to a composition of mortar in cases where the lining of the chimney is subjected to acid gases, such as we have in steel works, where we used the blast furnace and the exhaust is taken off by the chimney. The water vapor and sulphuric acid gas might have an effect on the chimney.

Mr. Mingle. MR. J. G. MINGLE.—The mortar material for the lining of chimneys for high temperature, is usually composed of a mixture of sand, cement, and fire clay, and I think you can use it in the chimneys you mention. Sometimes there is some kind of a water compound added, but instances of this kind do not come up very often.

MR. R. C. ALDRICH.—I have found in chemical work construction that mortar composed of cement and fire clay and no sand was pretty good. Of course it is not acid-resisting but it is better than ordinary mortar, very much better because the joints are very close. A simple fire clay and cement are mixed to about the same consistency as fire clay. The bricks can be made to a joint only about one-third to one-quarter of the thickness of the ordinary joint, so that the acid does not penetrate. **Mr. Aldrich.**

A MEMBER.—I would like to make the inquiry whether it is necessary in ordinary house construction to have flue linings in concrete chimneys? **A Member.**

MR. JOHN W. LOWELL.—The Building Ordinance of the City of Chicago allows the concrete block chimney flue 3 in. thick, and it is the only type of chimney that requires no lining. If a brick flue is to be used it must be one brick thick, and they require a clay type lining. I do not believe it is necessary to put in a lining. In fact there are a number of chimneys built for power plants where the chimney temperature is not over 500° and a concrete lining is used, not only as an independent wall inside of the stack wall, but in some cases merely as a guide wall, and good results have been obtained there. **Mr. Lowell.**

MR. A. N. TALBOT.—The author states that the chimney should not be used too soon after construction. I should like to ask how soon after the work is done it is practical to put such a chimney into service. **Mr. Talbot.**

MR. J. G. MINGLE.—It depends on different conditions. Sometimes a chimney, through necessity, is used 24 hr. after the work is finished, sometimes 6 months. I would recommend that it be left at least 30 days to give the concrete time to get its final set. **Mr. Mingle.**

REINFORCED-CONCRETE COLD-STORAGE WAREHOUSE AT BOSTON, MASS.

By G. H. Brazer.*

A reinforced-concrete cold storage warehouse has recently been completed in Boston for the Quincy Market Cold Storage & Warehouse Co. This building is designed for butter and beef storage and is located on T Wharf where there are ample facilities for unloading directly from steamers, railroad cars or teams. The building is 140 x 152 ft. in plan by 118 ft. high, and contains eleven stories together with a basement, forming a unit that will be duplicated in the future, as the business may require, the foundations for the next unit having already been put in place.

As has already been mentioned, this building is located on T Wharf, the westerly side of building being about 80 ft. from Atlantic Ave., which is the main water front street of the city. On the southerly side of the warehouse the old dock has been filled to form a roadway, thus giving access to the warehouse for teams and drays which may be backed up to the shipping doors. On the northerly side of the building there is a railroad siding connected with the Union Freight Ry. tracks running along Atlantic Ave., which permits unloading and shipping directly from the cars to platform at the level of the first floor. These railroad tracks are supported on heavy wharf construction. Immediately above the railroad tracks at the level of the second floor of the warehouse is another unloading platform extending out to within 10 ft. of the edge of dock. Heavy whip hoists are to be installed on the face of the building at about the level of the third floor, which will permit the unloading of beef from steamers or from cars on outer track. These whip hoists will lift the beef to this unloading platform at the second floor level where it will be taken directly into the building on trolleys.

On both the northerly and southerly sides of the building directly in the center are located service shafts containing elevators, stairways, pipeways, and space for conveyor. No attempt was made to maintain these service chambers at any constant temperature. The walls of these chambers are insulated on the side toward the cold rooms and the entrances into these cold rooms are doubly protected by air locks, that is, a small room about 9 x 14 ft. with refrigerator doors opening from both ends. Each one of these service shafts contains three freight elevators 8 ft. square and also a space 13 x 14 ft. for a conveyor which will permit of hoisting or lowering a great many sides of beef at one time. Extending out from these service shafts on the under side of each floor through the air locks and into the various rooms are trolley tracks upon which run small trolleys capable of carrying a side of beef, thereby doing away with all handling or trucking. On account of fire restrictions and also the possibility of the company wishing to use the building for purposes other than beef storage, the building is separated in the center by a brick partition wall insulated on both sides, running from basement through the roof.

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* J. R. Worcester & Co., Boston, Mass.

DISTRIBUTION OF SERVICES ON FLOORS.

The basement story is only 8 ft. high in the clear and is to be used simply for the storage of material such as pipe, lumber, etc., the only entrances to this basement being by means of trap doors through the first floor. About one-half of the first floor is used as a shipping room, there being a direct connection to this floor from the elevators, avoiding the necessity of passing through the service chamber. The remainder of this floor, with the exception of two small offices, is devoted to cold-storage purposes for quick freezing. A part of the second floor is occupied by coil rooms together with fans and motors for indirect refrigeration which produce extra heavy loads. The remainder of this floor not used for machinery is devoted to cold storage purposes. The third to eleventh floors are devoted entirely to cold storage purposes.

Over the eleventh floor of the service chamber there is another floor which is known as the machinery floor. This carries all machinery for elevators. The conveyor machinery is to be carried by steel supports of its own running down to the mat slab of the service shaft basement. Immediately north of the southerly roof house, two diagonal trusses are placed, one end of each being supported upon the wall of the service chamber and the other upon a column near the center of the building. Upon these trusses is carried the framework to support the sprinkler tank which has a capacity of about 30,000 gal. There is also a small brine tank supported on the wall end of the westerly truss.

DETAILS OF CONSTRUCTION.

The building is of reinforced-concrete construction with brick exterior walls resting upon a concrete mat which in turn is supported by spruce piles. Owing to the depth below the surface of a good supporting stratum it was found advisable to use piling, and as the weight of the building together with the live-load for which the floors were figured brought an extremely heavy load to the foundations, it was necessary to drive piles over the whole area of the building as close together as was practicable. This resulted in placing the piles on lines 1 ft. 3 in. apart in one direction and 2 ft. 2 in. apart in the other direction, staggering the piles. The piles penetrated the blue clay stratum and had an average penetration under the last ten blows of a 2200 lb. hammer falling 10 ft. of about 2½ in. Under the full live- and dead-loads these piles were calculated to carry a load of 14 tons each. The majority of the piles were cut at elevation +6.50 and the bottom of the concrete mat was at elevation 6.0. The total thickness of the mat was 3½ ft. Midway between top and bottom of this mat there were embedded under each line of columns in both directions running the entire length and width of the building four

$1\frac{1}{4}$ in. diameter rods; the object of these rods being to tie the structure together, rather than to form any reinforcement against bending of the mat.

On the northerly or dock side of the building the foundations had to be carried to a lower level, as the dock is dredged to 30 ft. below mean low water, at a distance of about 35 ft. from the face of the building. Starting at the first row of columns in from the northerly wall the foundations were stepped down until at the outer edge the bottom of concrete was 16 ft. below mean low water. From there out to the dock line a gradual slope was made which was rip-rapped in between the piles of the wharf construction above. Along this north side of the foundations between high and low water mark granite blocks were used to face up the concrete foundations, these blocks having been taken from the old dock wall.

The walls around the service shafts produce heavy loads on the foundations and with practically no loads from the interior of the shaft, it was necessary to increase the thickness of the mat foundation and reinforce the same very extensively in order that the loads from these shaft walls might be transferred to the piles under the center of the shaft. Upon the mat were constructed pyramid footings extending to within 2 ft. 6 in. of the first floor. These footings carried the column bases and were of plain concrete. The exterior walls were supported upon a continuous foundation resting upon the mat.

The floor construction throughout the building with the exception of the portion in the service shafts was of the flat-slab type with dropped panels around the column heads. With the exception of floors which carried heavy loads, these slabs were $8\frac{1}{2}$ in. thick, reinforced in two directions. The granolithic surface was placed a few hours after the main slab was run and was included in the slab thickness.

SEPARATE FLOORS INSULATED FOR DIFFERENT TEMPERATURES.

Upon the first floor slab there was laid insulation consisting of cork board 6 in. thick. Over this cork board was laid a 3-in. slab of concrete, the upper inch of same being granolithic surfacing and this slab was reinforced with $\frac{3}{8}$-in. diameter rods running in both directions and spaced 10 in. on centers. The second floor was insulated with cork board on both the under side and top side, the insulation for the under side, 2 in. thick, being laid on the forms before the concrete was poured and being dependent for its support upon its adhesion to the concrete. The upper insulation was 6 in. thick and was covered with a 3-in. wearing course of concrete and granolithic, reinforced as described for the first floor. The seventh floor was insulated on the top surface with 6 in. of cork board and a 3-in. concrete wearing surface over that, and the roof was insulated in a similar manner only here the cork board was about 8 in. thick. In all cases the insulating material on the floors was connected directly with the insulation on the walls, thereby forming a complete seal.

Exterior bays or bays adjacent to walls were made of shorter span in order that the same reinforcement as shown for interior bays might be duplicated. This worked out very satisfactorily, as the contractor wished to combine the rods for negative reinforcement and those for positive reinforcement into continuous lengths of rods wherever possible. Around the edges of all floor slabs a trimmer beam was used to transmit the loads to the columns, as the exterior columns did not have enlarged heads, but relied entirely on the loads being brought to them by the trimmer beams. These beams and exterior columns in all cases were kept clear of the wall to allow the wall insulation to pass unbroken. Trimmer beams were calculated as continuous, and in order to permit the reinforcement to run from one beam to the other, it was necessary to use a latticed column, therefore, all exterior columns were made up of four angles latticed together. In this way it was possible not only to carry the negative reinforcement through the column but also to maintain a constant section of beam by carrying the positive reinforcement through the column, thereby reinforcing the beam for compression as well as tension.

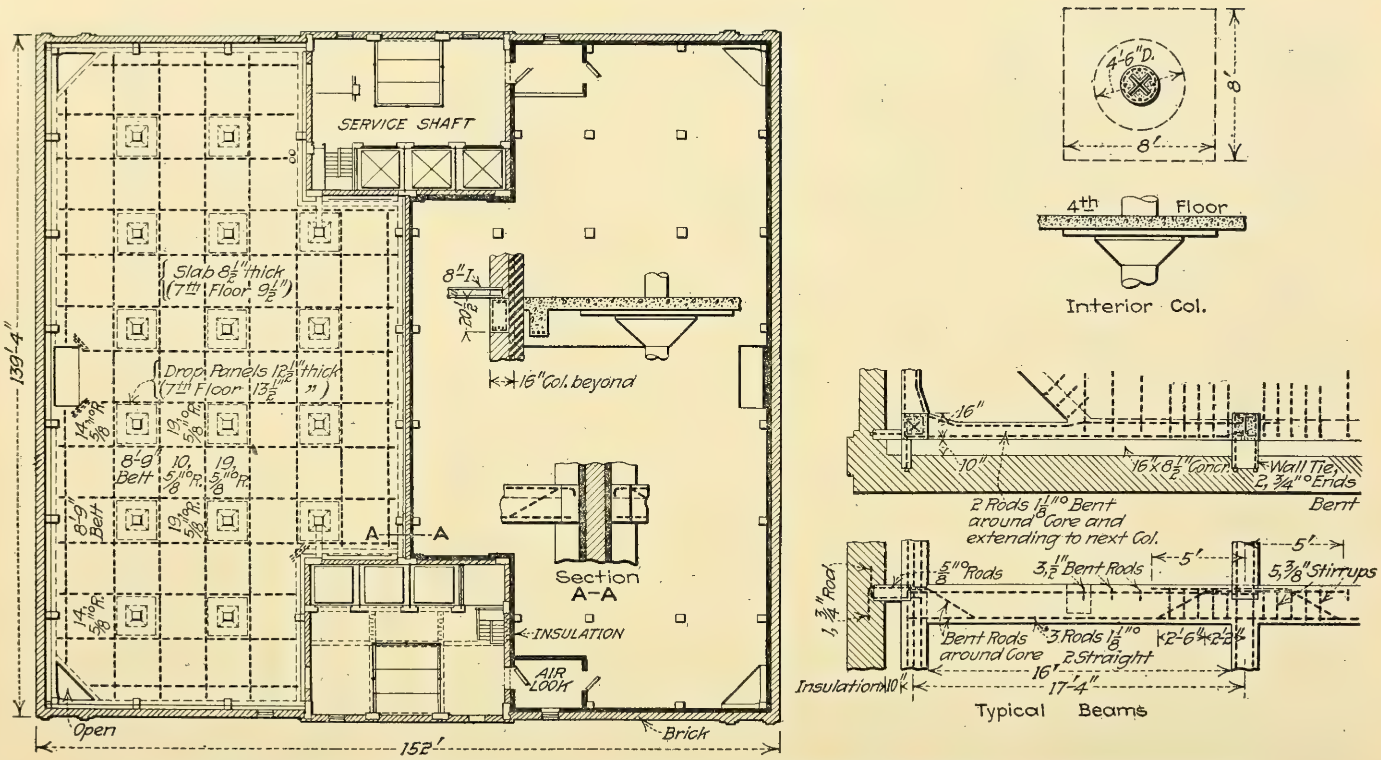
In a few instances around the service shaft it was necessary to run a beam into the wall surrounding the shaft, but in all cases these beams were made as small as possible, to reduce the interruption of the wall insulation to a minimum. The only important openings in each floor above the second, outside of the service shafts were triangular holes located at each of the four corners of the building and a rectangular opening at the center of the east and west sides. These openings are enclosed by 2 in. portland cement plaster partitions on wire mesh and channel-iron studs, and are used for ventilation purposes. On the under side of all floors sockets were embedded in the concrete for the support of the brine and sprinkler pipes and also for the support of tracks for the trolleys. The location of all of these sockets was carefully studied out beforehand and their positions were accurately located on the forms to which they were securely bolted before any concrete was poured.

The service shafts were so cut up by stairways, elevatorways and conveyor openings, it was impossible to use the flat-slab construction, and therefore beam and slab construction was adopted for this portion of the floors. It might be worth while to mention here that the construction of the building is greatly simplified by locating all stairways, pipeways and conveyors in an enclosed shaft and thereby leaving straightaway work for the main portion of the floors.

SPECIAL STEEL AND CONCRETE COLUMN USED.

A combination of a structural steel column with a reinforced-concrete column surrounding the same was employed for the interior columns, except in the three upper stories, where they were of reinforced concrete. All of the interior columns are cylindrical. The exterior columns are rectangular and are of structural steel fireproofed with concrete except in the three upper stories where they are of reinforced concrete. These exterior columns are placed inside of the outside walls and clear of them.

The type of interior column which was used in this building was devised by the writer in 1908, and first used on a similar building, and its principal advantage lies in the saving of floor space and its adaptability with the



DETAILS OR LAYOUT AND DESIGN OF TYPICAL FLOOR OF QUINCY REINFORCED-CONCRETE OLD STORAGE WAREHOUSE.

flat-slab construction. A few words describing the design of these columns might be of advantage. The loads from the roof, eleventh and tenth floors are carried by reinforced-concrete columns to the level of the ninth floor where this load is delivered into the structural steel core and travels through this core directly to the foundation. The ninth and eighth floors are carried on the reinforced-concrete casing surrounding the structural steel core down to the level of the seventh floor where in turn these are delivered into the structural steel core through a steel cap and thence these loads are carried to the foundation directly through the core. The seventh and sixth floors are carried on the reinforced-concrete casing down to the level of the fifth floor and there transferred into the structural steel core.

This method is continued to the bottom where there is a large cast-iron pedestal which receives the structural steel core together with the reinforced-concrete casing carrying the second and third floors. In order to obtain the full value of the structural steel cores, that is, to be able to figure them as structural steel columns, there is a complete separation made between the concrete and the structural steel core by means of enclosing this core in a sheet-iron casing; thus breaking all possibility of bond between the concrete and the core. The core is made up of four angles placed back to back, forming a cross, and in some of the heavier sections there are bars inserted between the angles. These angles are thoroughly riveted together and all rivet heads are countersunk in order that there may be no void spaces between the casing and the core, and also to prevent bonding effect.

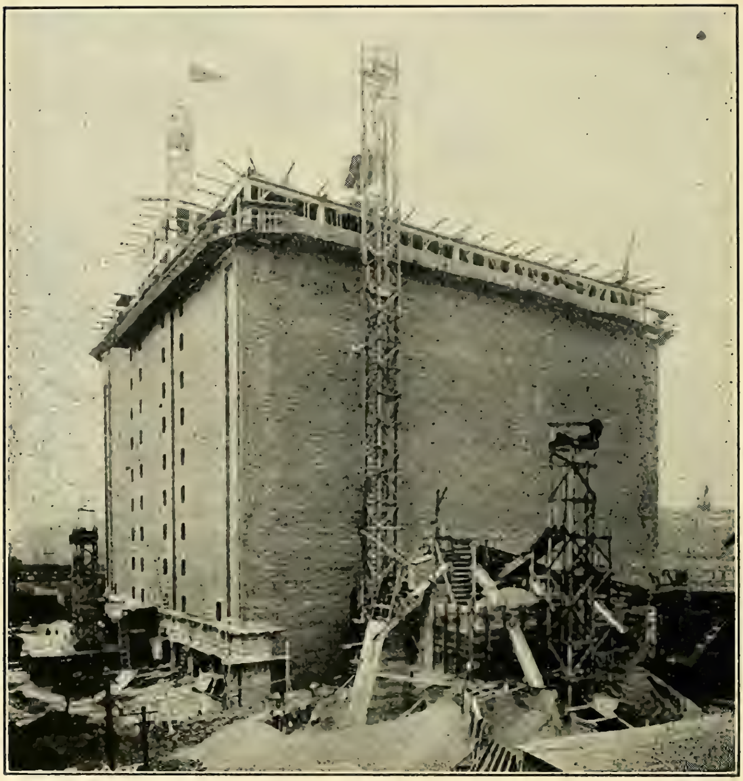
By changing the mix of the concrete and the amount of reinforcement it was possible to maintain a uniform diameter of concrete casing from the first to the ninth floors. The mix of the concrete was maintained the same in all columns of each story, although this mix did change in alternate stories. Considerable advantage was obtained in being able to maintain the same size of column from story to story, as steel forms were used in the construction and the number of these forms was thereby reduced to a minimum.

OUTSIDE WALLS INDEPENDENT OF FRAME.

One other interesting feature of the building is the fact that the outside walls are entirely independent of the framework of the interior and are self-supporting with the exception of a small withe extending from each exterior column at the floor line out into the brick wall. These withes were used solely for the purpose of offering a lateral support to the walls and in practically all cases they ran into the wall 8 in. and had rods extending from them both up and down into the brickwork.

The walls on the east and west sides of the building are designed as party walls, having the thickness prescribed by the law, as the Quincy Market Cold Storage & Warehouse Co. owns the property on both sides of this building and is proposing to utilize these walls in the future when buildings are constructed on both sides. These walls are 28 in. thick in the first story, 24 in. thick in the second, third and fourth stories, 20 in. thick in the fifth, sixth and seventh stories, 16 in. thick in the eighth and ninth stories and 12 in. thick above. They were constructed of a good sand-struck brick laid up in portland cement mortar.

The north and south walls are curtain walls and were made but 16 in. thick, except in the first story where, on account of the openings, for shipping doors, it was found necessary to use concrete 20 in. thick to obtain the required strength and for architectural reasons. The brickwork for these walls was a very hard, dense, glazed, face brick, which was adopted because of its density and very low percentage of absorption. These two walls, of course, will always be exposed to the weather, and being in an unprotected location, it was thought best to go to some expense in order to minimize the amount of moisture which might penetrate the wall. On the east and west walls this was not considered necessary, as it is very probable that within a year or so adjoining buildings will be erected which will protect these two sides. On the front and rear some attempt was made at architectural decoration but as the number of windows is limited and the position is more or less fixed by interior conditions, it is rather difficult to obtain a pleasing appear-



BUILDING THE QUINCY COLD STORAGE WAREHOUSE AT BOSTON.

ance. The coping and belt courses near the top of the building are all of pre-cast concrete stone, as is also the sign panel. This pre-cast work has a sand finish which was obtained from the sand molds in which the blocks were cast. The concrete portion of the wall in the first story was originally designed to be finished with a hammered surface, but for reasons of the owner, it was actually finished a smooth surface obtained by rubbing with carborundum.

The center partition wall and the walls around the service shaft above the first story were constructed of a sand-struck brick, and were 12 in. thick, the walls of the service shaft being supported on a reinforced-concrete framework, whereas, the partition wall was self-supporting from top to bottom.

In all cases with the exception of the front and rear walls where they came against the service shafts, the walls were kept clear of the column and floor construction at least 5 in. to allow the insulation to pass by unbroken. This complete separation of the floors from the walls was made in order to prevent loss of refrigeration. It also has the advantage of allowing the walls to work back and forth under changes in temperature and of preventing any strain on the floor construction while this motion is taking place. The floors will not be subjected to any marked temperature changes, as the temperatures in the building will remain practically constant and generally below the freezing point. The withes are constructed so as to offer as little resistance as possible to any motion which the wall may have in its natural plane.

Owing to the low temperature of the interior of the building and within the service shafts it was necessary to take all roof drainage through down spouts on the exterior of the building. The low points of roof were located on the northerly and southerly sides of building on each side of service shaft and carried through the parapet wall and thence down along the outside of the building through copper down-spouts which lead into cast-iron pipe about at the level of the second floor.

FEW WINDOWS PROVIDED.

In order to reduce the loss of refrigeration there are no windows provided in the walls surrounding the storage rooms, these areas depending solely upon artificial light. One window is provided in each air lock at each floor. These windows are fixed, triple glazed, steel sash. In each of the service shafts there are two windows at each floor level which are steel sash, single glazed and pivoted. At the north side of the building at the first floor level there are provided nine shipping doors and twelve on the south side, these doors all being double doors, framed of wood and metal covered.

The unloading platform on the second floor level on the northerly side of building was constructed with a steel framework upon which was laid a concrete slab. The object of using the steel framework in this location was due to the fact that this construction is largely supported on the piling and timbering of the wharf construction below.

The insulation throughout consists of a pure cork board which is manufactured by the Armstrong Cork Company. The outer walls above the second floor are covered with 8 in. of this cork board erected in portland cement mortar to which a waterproofing compound was added. The first, second, seventh floors and roof are also insulated, the object of insulating the seventh floor being to divide the building into two parts horizontally, so that the temperature of the rooms above may be considerably lower than that of the rooms below without affecting the temperature of the latter. As has been previously noted, this insulation is carried along the outside walls in an unbroken sheet, except for the small punctures where the withes pass through, and as the floor and roof insulation in all cases connects directly with the wall insulation, there is a complete seal about the building minimizing the external losses due to radiation. To further prevent loss of cold by opening and shutting doors into the storage portions, insulated air locks are provided at the entrances to all storage rooms.

This building is to be used for sharp freezing work where a temperature as low as 15° below zero F. may be maintained with ease. The refrigeration is obtained by a brine circulation system supplied by a central power station some distance away. The brine is refrigerated at this station and is pumped through underground pipes, thoroughly insulated, to the building. The refrigeration is a combination of the direct and indirect systems. The direct piping is suspended from the ceiling in each room, and consists of a large area of 2-in. galvanized iron pipe divided into convenient coils. The indirect piping is located on the second floor and consists of stacks of pipe divided into a large number of coils giving great latitude of temperature control. The air is forced over these stacks by means of large Sturtevant exhausts of the multivane type directly connected to Westinghouse motors and is distributed to the various rooms through flues or ducts. By means of this indirect system the whole or any part of the air from these coil rooms can be concentrated in any one room or distributed in any number of rooms as desired.

Six modern freight elevators of 4000 lb. capacity each, together with conveyors will handle in the most efficient manner the materials to be stored. Furthermore, a complete system of trolleys and tracks is being installed for distributing these goods to various parts of the rooms where they are to be stored. This equipment, together with the double railroad track and docking facilities on the northerly side, and the teaming or trucking facilities on the southerly side, give the building all of the possible advantages for handling goods in the most rapid and efficient manner.

In order to obtain the lowest possible insurance rates the building is sprinkled from top to bottom, the dry system being necessarily adopted. The electric light, power and heat are all supplied from the same central power station which supplies the refrigeration.

LIVE LOADS AND STRESSES.

The floors of this building were designed for a live-load of 250 lb. per sq. ft. with the exception of the space occupied by the machinery and coils on the second floor which was designed for 400 lb. per sq. ft. The superficial load on the roof was taken as 40 lb. per sq. ft. In designing the floor construction and also the reinforced concrete columns the rules and units as prescribed by the Joint Committee Report were followed throughout. The structural steel cores were designed in accordance with the requirements of the Boston Building Law, namely,

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The load on the piles, as has been previously mentioned was 14 tons per pile.

The construction of the building was commenced on or about Jan. 1, 1917, and was practically completed on Jan. 1, 1918. Almost six months of this time was consumed on the foundations which were extremely difficult owing to conditions encountered below the surface. On this particular location there has been a wharf of some sort, together with the buildings thereon since the latter part of the seventeenth century, and it was impossible to drive piling until this old construction had been entirely removed. Large areas of timber cribbing mortised and tenoned together were found as low as 4 ft. below mean low tide, and also innumerable old piles, and upon these there was considerable stonework all of which had to be removed before it was possible to drive the piling. The contractor at first attempted to remove this material by dredging, but it was eventually found to be impossible and the result was that a large portion of the area had to be enclosed by tight sheeting and pumped continuously in order to make any headway toward the removal of the obstructions. After the foundations were in place the floors were carried up at the rate of about a floor a week, and the walls, etc., followed along in due order. A very efficient material-handling plant was installed by the contractor which resulted in the saving of considerable labor.

The building was designed for the Quincy Market Cold Storage & Warehouse Co., Boston, Mass., by J. R. Worcester & Co., Consulting Engineers, and was constructed by the Turner Construction Co. of New York at a cost of about $625,000, exclusive of insulation, refrigeration, elevators, conveyors and trolleys. This latter part of the work the owners have done under their own engineering force.

Another cold-storage warehouse was constructed in 1908 for the same owners, this warehouse being used for the storage of butter. It was located along the same water front street as the one previously described, with railroad facilities, and also means of shipping by means of teams or drays. The general dimensions of this building were 155 ft. by 80 ft., and contained eleven stories. It was erected against another cold-storage warehouse owned by the same company, and was exposed on three sides.

Along the party wall were located the elevators, stairs, pipe shafts and air locks, these taking up a space the full length of the building, namely 155 ft., and projecting out from the party wall about 10 ft. This portion was separated from the storage rooms by an insulated partition running unbroken from top to bottom of building. The storage area was divided into three parts by a concrete wall insulated on both sides and an insulated partition.

The floor construction was of the flat-slab type and was figured for a live load of 250 lb. per sq. ft. The design of the columns and exterior walls was similar to the building previously mentioned.

The insulation of the walls and floors consisted of hermetically sealed cans of mineral wool, these cans being 5 ft. long, 2½ ft. wide and varying in thickness from 4 in. to 9 in. These cans were laid up on edge against the exterior wall in portland cement mortar after which 3-in. hollow terra cotta blocks were laid against them in portland cement mortar, and the whole of the interior being coated with a portland cement plaster, about ½ in. thick. The method of refrigeration was very similar to the building already described, but the temperature of this building was not required to run as low as the new one, the maximum degree of cold being 6° below zero F.

CORE CONSTRUCTION FOR REINFORCED-CONCRETE FLOORS.

By A. H. Bromley, Jr.*

In the early days of the development of reinforced concrete it was realized that the rectangular beam, with a width from top to bottom sufficient to develop its full compressive strength, was not an economical section, therefore, attention was centered on the study of the possibilities of the use of a portion of the slab at either side of the beam as constituting the compressive flange of the beam, and with the stem designed to care only for the protection of the steel and the secondary stresses to which it is subjected.

The first designs of this nature were carried out in connection with the use of slabs which were of such length of span as to require their being reinforced to carry the load between beams. A further development was the reducing of the width of the slab to a point where no reinforcement was required in it, thus effecting greater economy in the construction as the total steel required in the reinforcement of the beams is not increased by their closer spacing.

As in most cases the attempt to develop the idea was carried to extremes when considered from a practical and economical standpoint. Among these extreme ideas was included a system of cross T-beam construction, wherein the depth of the beam stem was reduced by reason of the load being divided up and carried by two beams at right angles to each other. This system also involved bearings being provided on all sides of the floor panel, and formwork, the cost of which was greater than the benefit derived.

The development of the one-way T-beam from a commercial standpoint, however, also proved very difficult owing to troublesome and expensive formwork involved. The first forms were made of wood, necessarily removable, thereby involving smoothed surfaces and rounded corners which were very expensive both in first cost and for re-use.

Another step in the development of a suitable form to reduce the cost of centering was the introduction of terra-cotta, clay-tile or plaster-of-paris blocks as fillers. These effected considerable saving over the wood form construction as the blocks permit the use of flat wood centering. This method, however, does not give maximum economy for the block is dead and useless weight that must be supported by the beams, columns and footings, and the freight and haulage charges often make the cost prohibitive.

The core type of reinforced-concrete floor construction in which pressed-steel cores are used to produce a series of parallel connected T-beams has come into extensive use and should be recognized as a standard form of construction. This type of construction is exploited principally by companies manufacturing specialties in pressed steel products and the forms are known by various names such as "Pressed Steel Cores," "Steelforms," "Metal Tile," "Floretyles" "Steel Tile," etc., and also known to many as the "Tin

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* Chief Engineer, Sales Dept., Berger Manufacturing Co., Canton, Ohio.

Pan System." This system provides a simple type of floor construction which may be supported by walls or beams of steel or reinforced concrete poured in conjunction with the floor system.

DESCRIPTION OF CORE DETAILS.

The various features that go to make up the system are as follows:

T-Beams.—The floor structure is in reality a series of connected T-beams presenting a flat surface on top. The flange, usually from 2 to 3 in. thick, forms the compression member of the T-section, and the web or rib of the tee, which encases the reinforcing steel, projects below the slab. These ribs are from 4 to 6½ in. in width, generally spaced from 24 to 31½ in. on centers and arranged from 4 to 14 in. in depth.

Ceilings.—In the great majority of cases a metal lath ceiling is secured to the bottom of the joists, which upon being plastered produces a flat ceiling surface. There is, however, a tendency to extensive use of the construction in buildings for factory or storage purposes, which are necessarily designed for heavy loads, and where the flat ceilings are not required. In these cases the cores are removed, and upon removal a concrete ceiling with a surface similar in appearance to a wood joist floor is produced.

Special Features.—The special feature in the core type of construction is in the use of a broad trough-shaped pressed steel core or center, placed in an inverted position, and supported by wooden skeleton centering. These pressed steel cores form the centering for the sides of the rib and the bottom of the flange, while the wooden supports form the centering for the bottom of the rib at the T-beam. The cores are varied in design and weight to meet requirements of either permanent or removable centering. Removable cores are usually made in from 16- to 22-gage and non-removable in from 24- to 28-gage material. Where necessary the cores are tapered in width to increase the width of rib adjacent to supports to make proper provision for the maximum shearing stresses or for requirements of negative bending moment. This is shown in one of the views. Suitable end caps are provided to close up the ends of the lines of cores.

Dimensions.—The dimensions of the cores are approximately as follows: Stock lengths 2 ft. 6 in. to 4 ft. Regular widths, 20 in. and 25 in., with special widths of 6, 8, or 12 in. and over, made for narrow spaces. Depths 4, 6, 8, 10, 12, and 14 in.

Shape of Cores.—The cores are made with sides slightly tapered and usually with a slight arch in the tops.

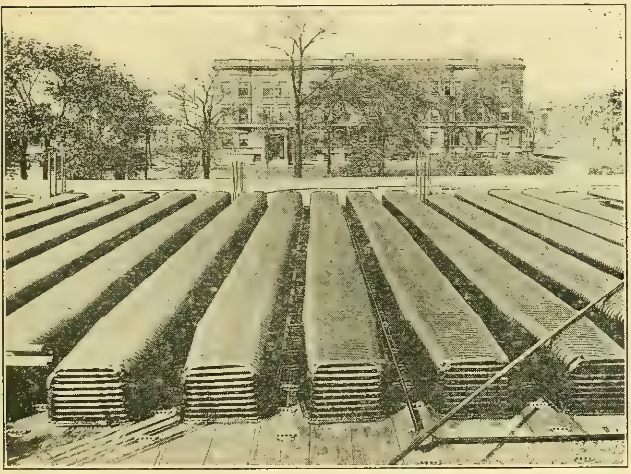
Beam Spacing.—It may be readily seen from the data given above that the skeleton wood centering to support the cores and form the bottom of the rib will require horizontal supporting members only 6 or 8 in. wide and spaced 24 to 31½ in. on centers, insuring great economy in material and labor as well as speed in erection of the carpenter work.

METHODS OF CONSTRUCTION.

There are two general methods of construction differing principally in the installation of the metal lath ceiling under the cores.

One method is where a light-gage ribbed expanded metal lath, or a stiff heavy gage lath is laid over the forms before the cores are placed. The concrete when poured engages the meshes of the lath in the bottom of the ribs, thereby securing it in position. It is generally found advisable, however, to secure wire to the lath, which wire is placed so as to extend up into the concrete rib, and thereby form a more secure fastening.

The other method provides for regular ceiling lath to be installed after the wood centering has been removed from under the construction. The cores are secured by lightly nailing them to the wooden forms, when wire hangers (consisting of a hooked end and straight portion about 15 to 18 in. long) are pushed through the holes previously punched through the tops or sides of the cores, and when concrete is poured these hook ends are securely held by it. After the concrete is poured and the centering removed, furring bars or rods are suspended under and at right angles to the ribs by securing them with these wires. The metal lath is then wired to the furring rods in the usual manner. Advantages are claimed for this method in that the ceiling plaster is kept away from the concrete, thus allowing an air space above the entire plastered ceiling. This eliminates discoloration of the white coat, which discoloration is usually in evidence where any type of joist construction is used which does not provide continuous air space. A further advantage of this method is that the centering can be removed for re-use.



CORES TAPERED IN WIDTH TO PROVIDE FOR SHEAR.

ADVANTAGES OF CORE CONSTRUCTION.

Among the decided advantages for this pressed-steel core floor construction are the following:

(1) Forty-five to 60 per cent saving in bulk of concrete over that required in slab construction of same depth; (2) saving in reinforcing steel in the floor by reason of light weight of concrete and core; (3) saving effected in entire structure, including beams, girders, columns, and foundations of the building by reason of small dead weight of floors; (4) weight of cores only $\frac{1}{30}$ as much as terra-cotta tile fillers for equal covering area; (5) bulk of cores only $\frac{1}{40}$ as much as terra-cotta tile fillers; (6) one core covers four times as much area of floor as one terra-cotta tile; (7) the forms produce tight centering, and eliminate waste of concrete; (8) small masses of concrete are used, thus reducing the immense quantities of moisture incident to larger masses of concrete, the drying out of which interferes with the heating of buildings during the first winter season.

Straight and fixed alignment of joists are obtained by lightly nailing the pressed-steel core in position, which alignment is necessary to develop the strength of the structure. Reid, in his book on "Concrete and Reinforced Concrete" (1907 ed., p. 525) referring to hollow tile fillers says: "In the construction of slab of this type some positive means should be employed to hold the tile rigidly in position until the concrete has been put in and set. The author has seen tile displaced as much as 2 or 3 in. during progress of construction. The strength of a floor rib would appear to be an uncertain quantity if even a single tile is displaced to any extent."

The danger of extensive damage by failure from defective material or workmanship is remote, as the floor consists of small units. Therefore, no large areas or excessive loads are dependent upon any one unit.

DETAILS OF DESIGN.

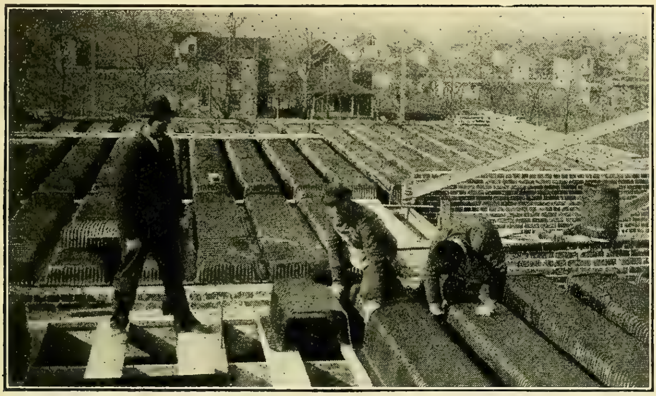
Refinement.—An inspection of the details of design shows it to be ideal from an engineering standpoint as all parts can be properly proportioned to stress and fireproofing requirements. This refinement of design eliminates the excess material used and the unnecessary weight imposed on the entire structure by other types of construction. The main structural parts of the building can also be fireproofed with the greatest ease in connection with the installation of the concrete work in the floors. The system provides for the design of concrete construction with a minimum quantity of concrete which adequately cares for the shearing and bond stresses as well as protection from fire to the reinforcing steel.

Depth.—The depth of the construction is determined by economical design of the concrete and reinforcing steel, or by the requirement as to ceiling heights. The flanges are usually designed as 2, $2\frac{1}{2}$, or 3 in. thick, making the total thickness of the floor structure range from 6 to 17 in. to which, of course, must be added the thickness of plastered ceiling and floor fill, if any, as well as floor finish.

Best Type.—Maximum economy is naturally attained in the use of 25-in. wide cores, it being evident that this provides the smaller quantity of concrete in ribs, while the quantity of concrete in the flange remains constant.

Concrete Frame.—When the core floor structure rests on concrete beams, economical design is also accomplished. In this case the ends of the cores are set back from the web or rib of the beam any distance to provide proper width of flange to develop the compression member. This upper flange will probably be of ample depth in that it includes the depth of both rib and the flange over the core.

Application of Formulas.—While the regular principles of design of reinforced concrete apply to this system of construction, owing to the thin and narrow slab and departure from straight lines in outline, the application of the usual T-beam formulas does not do justice to the section. One way to design quickly and within safe lines will be to work up a table of maximum resisting moments, based on the upper concrete compression flange value. These calculations can be readily made by applying the common theory of beams, which assumes that the stress in compression or tension is proportionate to the distance from the neutral axis. From this moment we can ascertain the greatest amount of reinforcing steel which can be used without overstressing the concrete for various depths and widths of cores. Having thus established the maximum value of the T-section we can proceed to design this type of construction by the usual rectangular beam formulas, with but the one restriction, *i. e.*, that we must not exceed in steel requirement the maximum allowable area previously determined. General rules by empirical formula as to proportion of stem to flange should not be construed to rule in this type of construction, provided we keep within the limits above outlined.

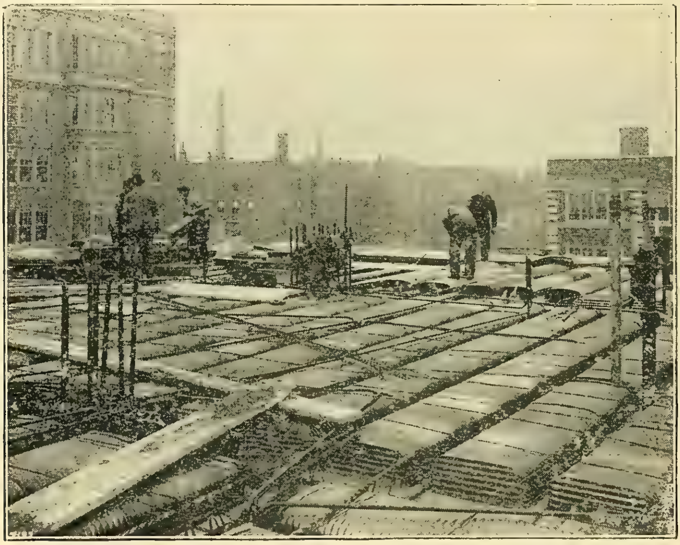


STEEL CORE SPANS TIMBER FALSEWORK SUPPORTS.

Published Tables.—The various companies exploiting these pressed-steel cores have published tables which may be obtained upon request. Undoubtedly these tables will be revised if standards are prepared by an authoritative body, which standards can be used generally for design and in practice. This will establish with building departments and commissioners a proper rating and basis for recognition of the system.

REASONS FOR INVESTIGATION AND STANDARDS.

Many architects, engineers, and companies supplying reinforcing materials work by empirical formulas or "rule of thumb" methods, without considering or realizing the great danger in such practice. There is also the tendency for some designers, when in close competition and when established regulations are not in evidence, to depart somewhat from good practice.



PLACING STEEL CORES FOR CONCRETE FLOOR CONSTRUCTION.

On the other hand, architects, engineers, building commissioners, and others, who are not familiar with the merits of the construction or its details, are apt to place restrictions which will bar its use in many places for which the construction is peculiarly fitted.

Believing that he voices the desires of the manufacturers in requesting standards and stating that there is need for the establishment of standards, the author suggests that the Institute prepare and recommend standard specifications. Therefore, this paper should not more than mention some of the questions which arise as to details of practice, and regulations with which manufacturers and users of this type of steel centering are confronted.

SEVERE CODE RESTRICTIONS.

Flange Thickness.—In exploiting the sale of cores we are confronted with codes in which slab thicknesses up to and including 4 in. are required. It does not appear right that this type of construction should be saddled with restrictions as to the thickness of the slab owing to the possibility of the use of materials which would be dangerous to any type of reinforced concrete construction. It is a well established fact that "lean mixtures and large size stone" will produce poor results in beams when rods are spaced as per usual standards, and in slabs where reinforcing steel is placed not over 1 in. from the bottom of the slab, therefore, regulations permitting a 2-in. slab based upon the use of 1 : 6 concrete with small gravel or broken stone should be termed as good practice where cores and short slabs are used. The surface of this slab is unlike the bottom of a beam, as it has no corners nor sides which can be attacked by flame, yet $1\frac{1}{2}$ or 2 in. concrete protection to steel in the bottom of beams is allowed by the majority of regulations.

Flange Width.—We are at times confronted with regulations allowing only three-fourths of the slab width between joists as a compression member. Since there is practically no slab stress there will be no simultaneous action as floor slab and joist flange in the plate of this type of construction, therefore, is it not reasonable to treat the entire width between joists as a compression member?

Steel Protection.—Shall the construction be considered a slab with core, and the protection under and at the sides of the reinforced steel in the joists be treated as in a slab with about 1 in. concrete protection, or shall the joists be considered as beams and the steel be protected by $1\frac{1}{2}$ in. of concrete? Also, what allowance, if any, can be made in this protection when metal lath and plaster ceiling is installed under the joists? And also, what further allowance can be made if an air space exists above the ceiling?

Determination of Bending Moments.—The author believes that bending moments can be figured as in beam construction, *i. e.*, simple spans $\frac{1}{8}$ WL, semi-continuous spans $\frac{1}{10}$ WL and continuous spans $\frac{1}{12}$ WL.

DISCUSSION.

Mr. Godfrey. MR. EDWARD GODFREY.—Mr. Bromley has set forth in a clear manner the qualities and the advantages of floors made with metal cores. He comes before the American Concrete Institute and the profession, not presenting extravagant claims and camouflaged tests purporting to show the wonderful tricks that this system plays on the laws of gravitation and statics, but with the suggestion that ordinary rules of design be applied. His suggestions are therefore deserving of careful consideration.

Wherein core construction differs from exposed beam and slab construction there should be allowances made for the differences. If metal lath and plaster is used under the cores and not under the beam and slab construction, not so much concrete need be used for fire protection of the steel. However, there should be concrete enough around the steel to grip the steel properly. This is a function of the diameter of the steel rods and should be gaged by that consideration.

The slab between two ribs 2 ft. or more apart could well be of a smaller minimum thickness than that required on slabs 4 to 6 ft. in span. These slabs, however, appear to need some sort of reinforcement to prevent shrinkage cracks, especially in construction where the cores are removed with the centering: in such construction reinforcement is also needed to insure the action of the slab as top flange in a concrete beam.

Mr. Bromley states that "where necessary the cores are tapered in width to increase the width of rib adjacent to supports to make provision for the maximum shearing stresses or for requirements of negative bending moment." It would appear that this would be necessary in all cases of continuous beams, or, in other words, in practically all construction. The negative bending moment over the supports needs to be taken care of, and a beam that is figured out to need the full width of concrete over the core for top flange at mid-span will need something better than the ordinary width of the rib for compression flange to take the negative moment at the support.

Mr. Bromley does not say how shear or diagonal tension is taken care of in these ribs. The writer believes, however, that, in this construction at least, foolish and defenseless stirrups and short shear members are discarded. The reason is doubtless accounted for by expediency. Developments brought about by expediency are often more effective and sure than those that are brought about by logical analysis. There is not an engineer, living or dead, who ever could or did explain the action of short shear members or stirrups, as commonly recommended to be used by practically all authors, with any explanation that would stand the test of the simplest logic. If closely spaced ribs need no so-called shear members, more widely spaced ribs are no more in need of them, and, since the principle of the rib is identical with that of the beam or girder, no beam or girder needs them. The expense and trouble of using them in ribs has led to their being discarded for this style of construction. **Mr. Godfrey.**

Any regulation governing rib construction such as this should require that where the unit shear on the concrete exceeds say 40 to 60 lb. one of the reinforcing bars be bent up, the bent-up portion reaching from about the quarter span point to the support, that is, not with a sharp angular bend but with a flat angle. The bent-up rod should be fully anchored beyond the edge of support. If there is another rib or beam in the same line, this rod should reach into that beam to the quarter point to act as top flange reinforcement to take the negative bending moment. Reciprocally, the rod of the other rib should reach into this one. This constitutes real, unassailable reinforcement for shear or diagonal tension. It can be explained and needs no apologies. It has been shown in tests to take the tension attributed to it: the same investigator found no tension whatever but compression in stirrups.

Mr. Bromley. MR. A. H. BROMLEY.—It is usual, I think, to keep the shear within 60 lb. Personally, I favor a shear stirrup of about 60 lb. with bent bars.

Mr. Hatt. MR. W. K. HATT (*in the chair*).—I would like to ask the author, concerning the motion that a committee be appointed to standardize this construction, in what manner it is so peculiar that the present standards which are drawn up for construction will not apply to it.

Mr. Bromley. MR. A. H. BROMLEY.—The thickness of the slab is really not a determined amount. Some parties claim that it should be 4 in. thick, some 2, some 3 and some 2½ in. There seems to be nothing to cover that. Some people say you can use 2 in. on top of a tile floor or a plaster block floor, but cannot use it with cores because you do not have the excess strength of the tile or the assistance of the tile. There is also the question whether the dropped section is a slab core or whether it is a beam. If it is a slab core, slab reinforcement can be used; this would be about 1 in. from the bottom, giving possibly about ¾ in. protection. If it is a beam, the rods would have to be up about 2 in., giving 1½ in. protection, the usual beam practice.

Mr. Hatt. MR. W. K. HATT.—The Institute already has a Committee on Reinforced Building and Concrete Laws. It might be well to refer this style of construction to the committee to determine if the rules already drawn up govern this construction or if others need be reported. Is that satisfactory?

Mr. Bromley. MR. A. H. BROMLEY.—Perfectly.

THE CHICAGO & NORTHWESTERN RAILWAY COMPANY TERMINAL ELEVATOR.

BY F. C. HUFFMAN.*

The Chicago & Northwestern Ry. Co. at the close of the year 1917 leased its new 10,000,000-bushel terminal elevator to the Armour Grain Company for operation. This grain elevator is located on the west bank of the Calumet River, near 122d St., in the City of Chicago, Ill. It is the largest and most completely equipped grain elevator in the world, having all the latest type of grain-handling machinery and apparatus needed for taking care of grain on a very large scale.

The plant is so situated that grain is received both by rail and water transportation, as well as being shipped out in like manner. The daily receiving capacity by rail is 1,296,000 bushels; the daily shipping capacity by rail is 1,296,000 bushels; the daily receiving capacity by boat is 480,000 bushels; the daily shipping capacity by boat is 1,440,000 bushels; all of the above operations may be carried on simultaneously. The handling capacity of the elevator relies upon the receiving capacity, which is 1,776,000 bushels per day, or as expressed in car units, would amount to over 1,000 cars per day.

The main structure consists of track shed, work house, storage house and river house, all of which is connected in one unit and built of concrete reinforced with steel, with the exception of the cupola which is built of structural steel and supports curtain walls of concrete 2 in. thick, reinforced with No. 9 wire mesh. These curtain walls are built of gunite, placed by the cement gun method.

As it is the intention of the writer that this paper is to be submitted to those who are greatly interested in concrete work, the method and manner of carrying on this work will be taken up in detail.

It is quite necessary when undertaking the construction of such a large structure to have a large and well-laid-out concrete plant which will economically carry out the work. The plant consisted of an elevated railroad track for receiving sand and gravel in hopper-bottom cars which were dumped from trestle into hopper from which a belt conveyor carried the material up an incline of 20°, to the horizontal, to a height of 40 ft., where it was dumped and uniformly distributed by a traveling tripper into a large storage bin 200 ft. long. This bin is divided into a compartment for sand, holding 1,000 cu. yd., and the remaining portion for holding gravel to the amount of 2,000 cu. yd. The bin has a sloping bottom and has a belt conveyor running the entire length underneath, which receives the material from the large bin and carries it to a cross belt which again elevates the material to a height of 40 ft., where it

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*Resident Engineer, C. & N.W. Ry. Co., Chicago.

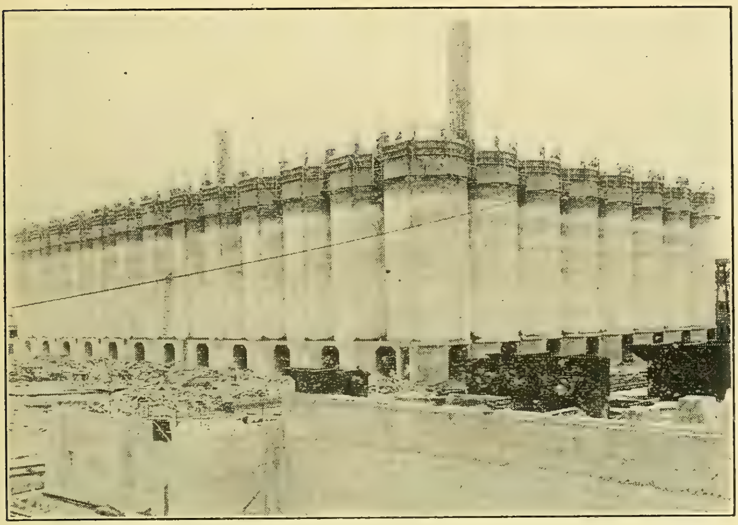

FIG. 1.—GRAIN ELEVATOR UNDER CONSTRUCTION. MOVABLE FORMS AT THE TOP OF BINS.

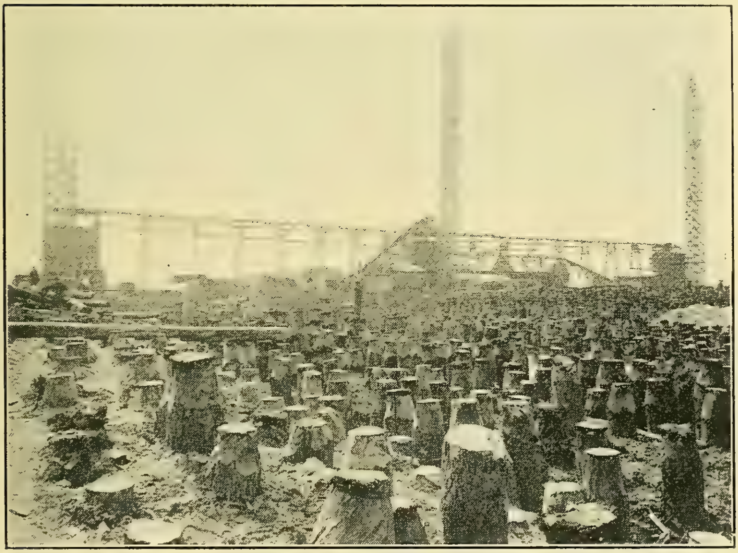

FIG. 2.—PILES FOR THE FOOTINGS OF THE ELEVATOR IN FOREGROUND. TRESTLE AND CONCRETE PLANT IN BACKGROUND.

delivers it to a long belt running parallel to the building. This belt delivers to three separate mixing plants equally spaced near the side of the structure.

Each mixing plant has a sand hopper of 40 yd. capacity and a gravel hopper of 80 yd. capacity located directly over the mixer. The material is delivered to the mixer through measuring chutes. The cement in most cases was taken directly from cars which were brought opposite each mixer on a material track. A 600-bbl. cement shed was built at each mixer plant to store cement in case cars were not regularly delivered. Water was supplied from the river by the use of a motor-driven triplex pump, automatically controlled by cut-off switch, at a maximum pressure of 80 lb. A one-yard electrically driven mixer was installed at each plant. After the concrete was mixed it was dumped into hoisting buckets at each of the three plants and hoisted to proper height, where it was delivered to distributing chutes. The distributing chutes had a swivel leg and balanced cantilever discharge spout operating in a radius of 100 ft. This method of handling the concrete was used for all the foundation slabs, basement walls and bin floor slabs. Above these, it was sliding form work and the "buggying" method was used. Each of the concrete plants was able to deliver 1,200 yd. during a period of two 10-hr. shifts. When operating at capacity a batch of concrete was turned out every 47 sec.

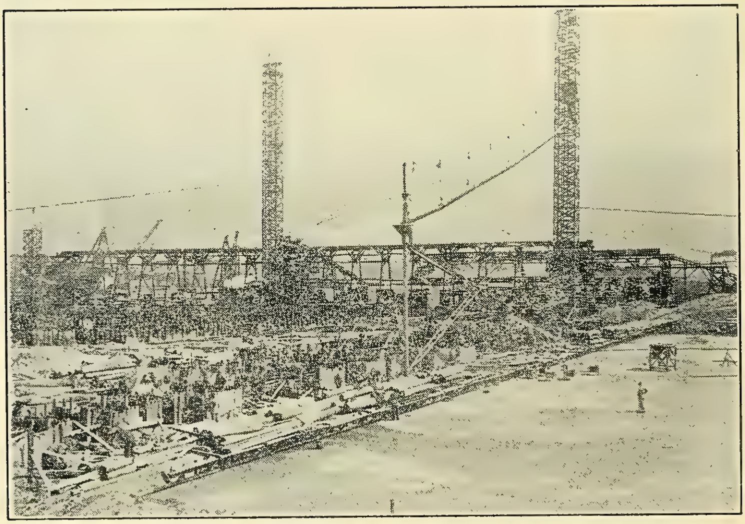

FIG. 3.—CONCRETE MATERIAL PLANT AND CONCRETE BEING PUT ON THE FIRST SLAB OF TRACK SHED.

All conveyors, hoists, mixers and machinery were electrically driven by current brought in at 12,000 volts and stepped down to 220 volts at a temporary transformer building on the site.

The foundation for the structure consists of 18,200 timber piles of an average length of 36 ft. below cut-off, spaced 2½-ft. centers. After the piles were cut off the concrete mattress was poured over the entire area. This mattress consisted of gravel concrete, heavily reinforced with steel bars, varying in thickness from 1 ft. 6 in. to 2 ft. 4 in., as requirements demanded. As soon as a section of the mattress was completed, workmen began placing of the stationary forms for the basement story. This story is used for the machinery and belt conveyors used in conveying the grain from the track shed to the work house or river house and receiving same from the river house, storage bins, and delivering to work house. The walls and piers in this story are of massive design owing to the enormous load they must support. In the storage house and river house the forms were arranged that the bin bottom slab and walls were poured so as to make a monolithic structure.

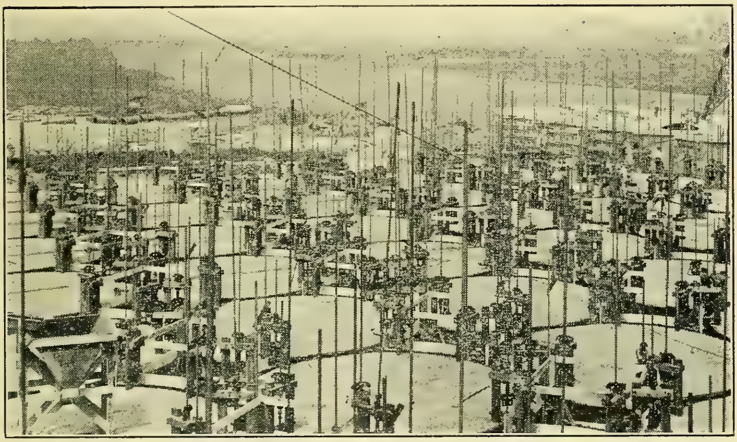

FIG. 4.—LOOKING OVER THE TOPS OF THE BINS SHOWING THE JACKING APPARATUS.

The concreting of the circular bins was next in order of procedure and is very interesting. The bins of the storage house, 104 in number, are 105 ft. high, 22½ ft. in diameter of the center line of walls, which were 7 in. thick and reinforced horizontally with flat bars. The forms, as shown in one of the views, consisted of an outer and inner wall section built of 4-in. beveled vertical staves, 4 ft. high, and held together by yokes. There were eight yokes to a bin, each yoke having attached thereto a lifting jack of the "monkey motion" type. This type of jack was especially designed for this job, and operated upon a 1-in. pipe which served as the vertical reinforcing for the walls. This jack operates with a lever operating through a vertical arc and eight were used on each bin form. The vertical pipes were spliced with butt joint and filler which permitted the jacks to operate without any interference with the joints or in placing additional lengths of pipe.

The pipes were marked at 6-in. intervals, which gave the workmen on the jacks evidence when they were carrying the forms up in a level position and also the amount they were required to raise at any one time. These forms were so connected by vertical rods and cast slides that the raising of one form did not disturb the adjoining forms. Each bin form was lifted in turn 6 in., the workmen making the round in regular order, and the steel placers following closely behind, after which the concrete was poured, making the work a continuous performance. These 104 bins were run up 105 ft. in 17 days, working two 10-hr. shifts per day. The 24 bins of the river house were run up a distance of 95 ft. in 11 days time, working two 10-hour shifts per day.

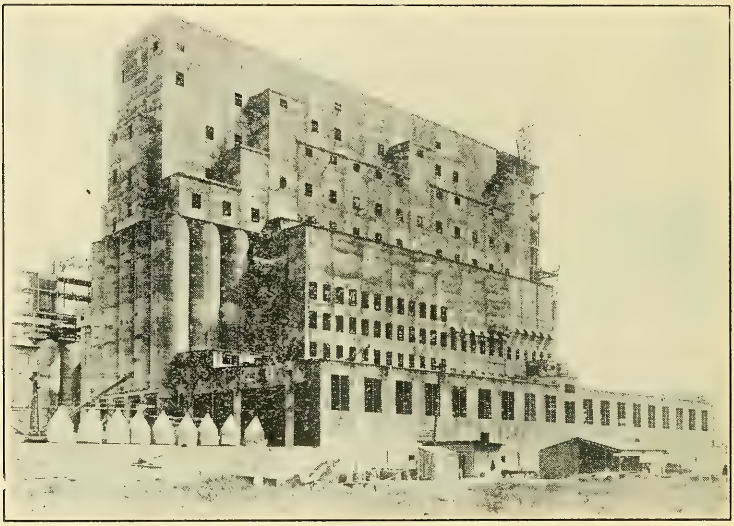

FIG. 5.—THE FINISHED REINFORCED-CONCRETE GRAIN ELEVATOR OF THE CHICAGO & NORTHWESTERN RAILWAY AT CHICAGO.

Above the tops of the bins, the cupolas over the different units were constructed of structural steel frame enclosed with 2 in. curtain walls reinforced with No. 9 wire mesh. These were placed by the use of the cement gun. All roofs and floors were constructed of reinforced concrete.

There were 62,000 cu. yd. of cement, 1,750 tons of reinforcing steel, 2,000,000 ft. of form lumber, 94,000 bbl. of cement consumed in this structure.

Owing to its enormous size and wonderful capacity for handling grain, this grain elevator is attracting world-wide attention and stands as one of the great monuments representing this grand and glorious concrete era.

PROBLEMS IN THE DESIGN OF A CONCRETE FACTORY.

By N. M. Loney.*

A large factory recently built for the American Can Co., at Maywood, Ill., involved a number of problems in the design of a manufacturing establishment. The author's work has been confined, for the last twelve years, to the design and equipment of factories for similar purposes and a number of features applicable to similar industries have been evolved which are exemplified in this particular factory and which, it is thought, may be of sufficient general interest to justify a short account of this construction and the designs leading up to it.

In general, the floor loads required in such factories may be classed as those suitable for light manufacturing and, in common with similar industries, the slow-burning type of mill construction was formerly considered the standard, but the limitation in practicable spans and excessive vibration led to the adoption of the beam-and-girder type of reinforced-concrete construction with removable brick spandrel walls, and this type was adhered to for several years until the further advantages of the beamless ceiling construction led to the adoption of the flat-slab type although, at that time, the methods of calculation of the strength of same were still in an undeveloped state.

In buildings lately built, there are now being used, except in special cases, either the two-way or four-way type of flat-slab construction with depressed panels and spans up to 22 ft., with the greatest of satisfaction, and the developments of our later design have been along the lines of increasing the usefulness of the buildings by special arrangements for better natural ventilation, light and heat and service features, etc., all of which are well exemplified in the Maywood factory.

The limitation of the site required a three-story and basement building for the capacity intended to produce standard packers can at the rate of 150,000 per hour, with a further reserve space for 25 per cent additional manufacturing capacity and for the storage, at one time, of approximately 50,000,000 cans and car space for simultaneous loading with finished product and the unloading of raw material, of 60 cars together with the necessary sorting yard to take care of these cars and approximately the same number of additional cars from the adjacent and older factories. This large number of cars is necessitated by the fact that this is a seasonable business and the slogan of the trade is "Service." The proposition was further complicated by the known fact that the adjacent main railroad, serving the plant, was shortly to be elevated, although, for the present, it remains at the original grade.

The shipping problem was solved by adopting a U-shaped building with manufacturing done on the top floor, in the space connecting the legs of the

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[*] Chief Engineer, American Can Co., New York City.

"U" of cross-section. The two legs of the "U" were designed to be used for storage of cans and the space between utilized for four lines of twelve railroad cars each, separated by platforms and covered overhead by an asbestos protected metal canopy roof, supported on steel trusses which also served for the support of the main conveyor system. Two lines of twelve cars each, were located on the outside of one leg of the "U" and these two tracks were extended to serve the coal bins of the power plant by gravity.

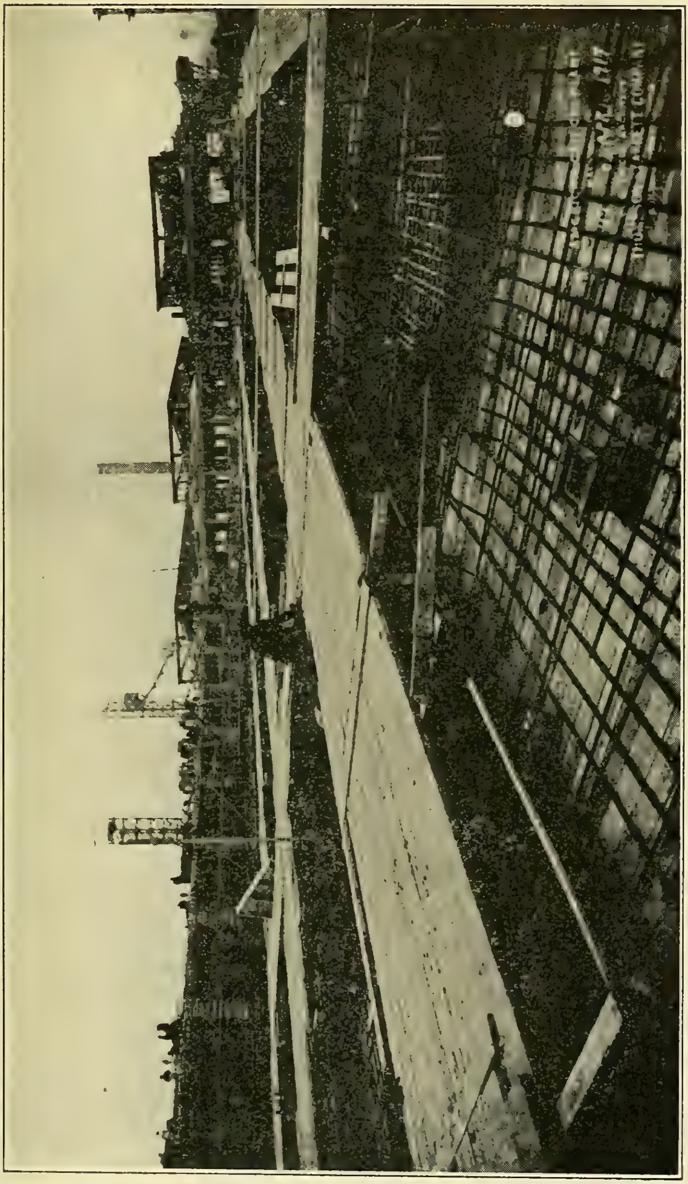
The building being so close to the main railroad, it was necessary to bring the above tracks in at an elevation of about 12 ft. above the normal grade to prevent a dangerous down grade leading into the building, after the elevation of the adjacent main railroad. These tracks are, for the present, reached by a rather steep temporary upward grade. The outside of the other leg of the "U" being the street side, was utilized for the installation of eight inside motor truck loading stations in the basement, the floor of which, however, is only a few inches below normal grade, the truck floor level loading platform for the trucks being reached by ramps from the basement and directly by elevators from the other floors.

The tracks mentioned above are supported entirely on reinforced-concrete trestles incorporated in the construction of the building and the space below utilized for the storage of the various raw materials entering into the construction of cans.

The total floor space is approximately 520,000 sq. ft. The building is especially arranged for the automatic deliveries of cans from the point of manufacture to the storage bins or cars. Conveyors travel in the monitor of the roof over the track court between the legs of the "U" traveling through dormers built over the sloping side of the track court roof for this purpose. All of the above conveyors have runways of steel gratings located below them for the conveyor attendant.

The use of saw teeth in former constructions has not been found to be well adapted for ventilation although the light effects are very desirable. It has been found by some study that by having the monitors run east and west during the summer season when the direct sunlight is objectionable, it is practically confined to a narrow band with an approximately fixed location upon the floor, and a little study enables the operations to be so located as to avoid the band of sunlight while, if the monitors are run north and south, every point on the floor is at some time subjected to the direct rays of the sun.

Formerly roofs, for similar purposes, were designed with a unit of three bays, with a monitor one bay wide with an adjacent bay with sloping roof on each side. The arrangement here enables us to secure practically the same amount of light with one-third less glass surface in the monitor, on account of the reflection from the inside of the sloping-roof surfaces. The type of roof enables us to construct a floor, of any desired area, independent of any side light, and aids materially in securing a well ventilated and airy work room as well as to a certain extent lessening the noise from the rolling cans which in a flat-roofed room make it almost impossible to be heard. A gridiron system of steel beams supports the power transmission equipment; by means of



VIEW SHOWS TYPE OF SPECIAL ROOF ADOPTED IN MAYWOOD FACTORY.

special clamp hangers can be fastened at any point, without the necessity of drilling holes. In later constructions, now under way, concrete beams with lines of continuous inserts on both sides and bottoms of same are being used instead of the steel beams here shown.

Nicely finished service rooms are conveniently located for each sex, containing steel lockers and high-grade plumbing fixtures including individual wash basins, shower baths, etc. A completely equipped dining-room and kitchen are also provided where meals are served at cost. There is a so-called first aid department, finished in enamel tile, containing a doctor's office, operating room, waiting room and convalescent room, having a nurse in constant attendance and regular calls from a physician.

The contractor encountered an unusually severe winter, the coldest for about 18 years in Chicago, the thermometer dropping below zero early in December, necessitating the dynamiting of most of the excavation ahead of the three steam shovels employed and, as the winter proceeded, more and more elaborate precautions against freezing had to be resorted to. In addition to heated bins, for the storage of aggregate, it was found necessary to provide a heated car shed for the preheating of material in cars and enable it to be handled. An ample steam supply was maintained ahead of the concreting to insure the removal of frost from below the footings and from the forms. Tarpaulins were provided to immediately cover the concrete after pouring; a patrol of numerous thermometers was maintained night and day by the writer's organization to make sure that heat was kept up by the salsmanders, used under freshly poured concrete until same had set.

The first supported floor forms were built in such a manner as to bear on the footings only, thus avoiding any settlement or upheaval, due to frozen ground.

The work included girders of 40-ft. span, poured in zero weather, the work proceeding at times, both day and night, and practically no damage was done by freezing, which leads the writer to conclude that, if suitable precautions are exercised, concrete may be safely utilized at any temperature at which men can be persuaded to work.

Great credit is due to the Thompson-Starrett Co. for their accomplishment of what, we believe, constitutes a record for winter construction.

The reinforcement was designed by the Corrugated Bar Co. on their well-known two-way system, practically according to the Chicago Code but with modifications adapted by the writer as being more in line with the conservative practice which he has always followed in flat-slab design, this consisting principally in increasing the minimum thickness of slabs and depressions and the lengthening of laps of the reinforcing bars.

The company with which the writer is connected now has in service well over 3,000,000 sq. ft. of flat-slab construction, the early examples of which were largely empirically designed but, in a general way, with considerably more steel and concrete than was customary with either early designs of this type and the results of the more exact knowledge have been to decrease, rather than increase, the amount of steel and concrete used.

REINFORCED-CONCRETE FLAT-SLAB RAILWAY BRIDGES.

By A. B. Cohen.[1]

The art of reinforced concrete had its inception in Europe and flourished there for a number of years before taking root in this country. Although the development here has been of the highest order, there are foreign precedents for a major portion of the various uses for which we have structurally combined steel and concrete. The style of our American design has differed from that of the European to meet the widely different economic conditions of labor and of materials. With limited labor and abundant resources—the reverse has been true in Europe—our efforts have been concentrated, in a general way, toward the simplification in design and expedient methods in erection. The results are exemplified in no greater instance than in the development of the girderless floor, better known as flat-slab construction, where the simplicity has been so perfected as to produce the extreme European effort in conservation of materials without the excessive expenditure of time and labor incident thereto.

One has only to mention the recent astonishing record, seemingly incredible, made by the use of the flat slab in the erection of the Brooklyn Naval Storehouse, to prove conclusively its remarkable utility. This eleven-story building frame covering a ground area of 180 x 260 ft., or 1½ acres, was completed in 14 weeks by 500 men working one shift a day. The building, entirely fireproof, is capable of holding 70,000 tons of supplies.

Other building achievements could be mentioned to show the extraordinary development and utility of the flat-slab system for light building loads. It is the object of this paper, however, to show that the flat slab can be utilized, with equal effectiveness and added advantages, in carrying heavy railway loadings, in the construction of viaducts, and especially bridges of lesser magnitude, where the required span length is not prohibitive. All forms of concrete construction have this limitation.

The principal advantages of the flat slab compared with all other forms of reinforced concrete and other fireproof construction are embodied in the simplicity of both the formwork and arrangement of the reinforcing steel. The first cost of construction has been so reduced thereby as to put structural steel, in competition with the flat slab within its limitations, substantially out of consideration; furthermore with the concrete construction lower maintenance charges prevail and greater permanency is obtained. The simple arrangement of the reinforcing steel, laid over a practically unbroken flat surface, insures a more positive placement of the reinforcing bars than the general beam and slab design in concrete.

In addition to these general advantages of the flat-slab construction, the salient advantages resulting from its adaptation to railway structures will be brought out in subsequent description and illustrations of actual examples. There is, however, one outstanding feature of the flat-slab system which in the writer's opinion is of most vital importance in reinforced-concrete construction. By reason of its uniform cross-section and continuity of the reinforcement, there is no other type of reinforced concrete that is better proportioned to resist shrinkage and thermal changes. Structures of the flat slab have been built in surprisingly great lengths without the incorporation of a single expansion joint and have successfully resisted the very severe strains of these stresses. By the insertion of an additional amount of reinforcing steel across construction joints, a constant tensile resistance can be maintained which has the effect of preventing cumulative action of the stresses at any particular section; the strain is distributed uniformly throughout resulting in an infinite number of minute cracks that do not impair the strength of the structure.

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[1] Assistant Engineer, in charge Concrete Design, the Delaware, Lackawanna and Western Railroad, Hoboken, N. J.

Our experience does not extend over a sufficient length of time to ascertain definitely what effect the repeated action due to temperature changes will eventually have on the strength of the structures. However, very close observation of existing flat-slab structures, in service from three to six years, have disclosed no deleterious effect due to these causes. The minute cracks found were of no greater concern than those developing on the tension side of a beam long before the steel has reached full working stress.

By way of comparison in this regard, to show the difficulties encountered in other types of concrete construction, consider the special arrangements in the manner of expansion and sliding joints that are necessary and not always efficacious in large concrete-arch viaducts or in viaducts of the column, beam and slab design. In the viaducts consisting of a series of large main arches surmounted by transverse spandrel walls supporting a floor system, the vertical movement of the heavy arch ring, for a rise and fall of temperature, is transferred to the floor system. This very appreciable vertical movement must be resisted by the comparatively light floor in addition to its own changes in a horizontal plane. In the case of the beam and slab design the constituent members have different sections and therefore offer varying degrees of tensile resistance. There arises the difficulty of transferring the movement from the larger through the smaller members, as from the deep beams through the thin slab, which is not always satisfactorily controlled.

SOO LINE TERMINAL AT CHICAGO.

In 1912–13 the first and so far the most extensive application of the flat-slab system for carrying railway loadings was made in Chicago with the erection of the Soo Line Freight Terminal.

The yard area required for this improvement amounted to 18½ acres, comprising eleven city blocks located near the business and manufacturing centers. This entire layout for handling freight is carried on an elevated structure to meet the municipal requirements that no grade crossings should exist. Deck construction gave the greatest possibilities of storage development, making available 520,000 sq. ft. on the ground surface underneath the deck for this purpose. The flat slab showed advantages of lower cost, lower maintenance and greater permanence as compared with structural steel. Fig. 1 is a general view of construction which gives an excellent idea of the size of the undertaking and some understanding of the manner in which the work was carried on. From a railroad point of view the outstanding feature is the possible flexibility in the track layout since the structure is designed to carry any arrangement of tracks on 12-ft. centers. This was obtained with very little additional cost over a fixed position of tracks and driveways. It was in this structure that those responsible for the design decided that no expansion joints were necessary and their judgment seems to have been justified.

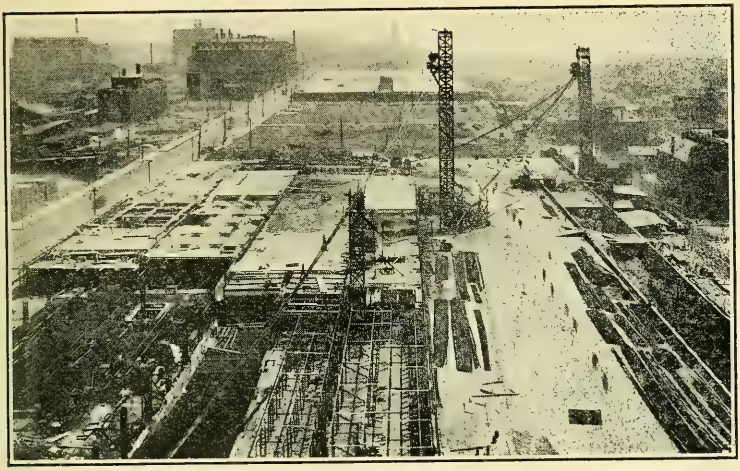

FIG. 1.—THE SOO LINE TERMINAL IN CHICAGO—A PIONEER FLAT-SLAB RAILWAY STRUCTURE.

D., L. & W. R. R., BUFFALO, N. Y., TERMINAL.

The highly satisfactory results obtained with the flat-slab system at the Soo Line Terminal prompted its consideration and adoption by the Delaware, Lackawanna and Western Railroad in the recent construction of a viaduct approach to the station of the new terminal improvement at Buffalo, N. Y. (See Fig. 2.) The viaduct, 154 ft. in width and 1070 ft. in length, supports a structural steel trainshed, platforms, and seven tracks on ballasted floor. This is shown in plan and cross-section (Fig. 3). For reasons analogous to to those cited in the first example, deck construction was admirably adapted to the maximum development of full terminal facilities in a very limited area. This new layout is located alongside the Buffalo River. Docking facilities are for Great Lake steamers which can be unloaded directly under cover of the slab where storage and other shipping facilities, including the express companies, are available. Two tracks are located on the dock level which connect by means of the subway and ramp tracks, with the main line tracks on the upper level. Passenger traffic is discharged on the upper level precluding interference with other station appurtenances best located on the ground level.

The entire structure is supported on timber piles and gravity footings. A precautionary measure was taken to prevent possible movement of piles by connecting all the piers in both longitudinal and transverse direction with reinforced-concrete beam structs or ties, thus insuring lateral stability. Single drop panels connect corresponding columns of bents 1 and 2 to reinforce the end section to take impact transferred from the bumping post which is anchored to the slab. The perfectly flat unobstructed floor simplified the waterproofing treatment which consists of a membrane composed of two layers of cotton cloth saturated and applied with hot asphalt and protected by a cover of asbestos paper and two $\frac{3}{4}$-in. layers of asphalt mastic. Only one expansion joint was provided and this placed at bent 31 where the slab begins to narrow down from the seven-track to the two-track width at the easterly end. The joint was deemed necessary here for the reason that the narrow section would not offer the same tensile resistance to temperature changes as would the wider section. An accumulation of stress might reasonably be expected somewhere in the narrow section if no expansion joint were provided which might result in cracks of sufficient magnitude to impair the strength of the viaduct.

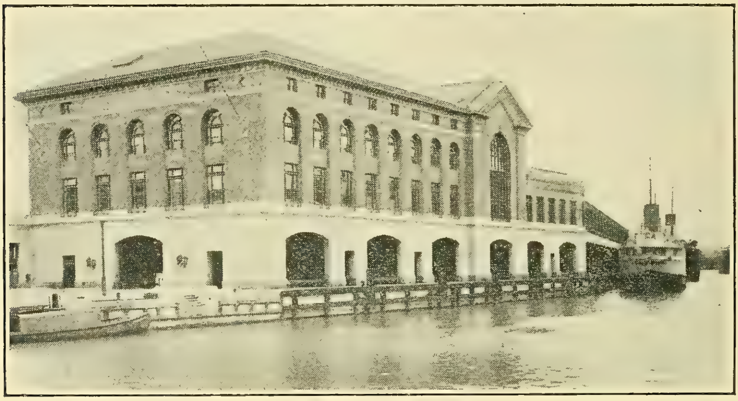

FIG. 2.—LACKAWANNA RAILROAD'S BUFFALO TERMINAL.

It might be noted here that the only cracks so far developed have occurred in the end panel under tracks 1 and 2 between bents 24 and 25. The principal crack starts at a point where the center line of track 3 intersects the side of the drop panel of column in bent 24 and bows out slightly in the arc of a circle

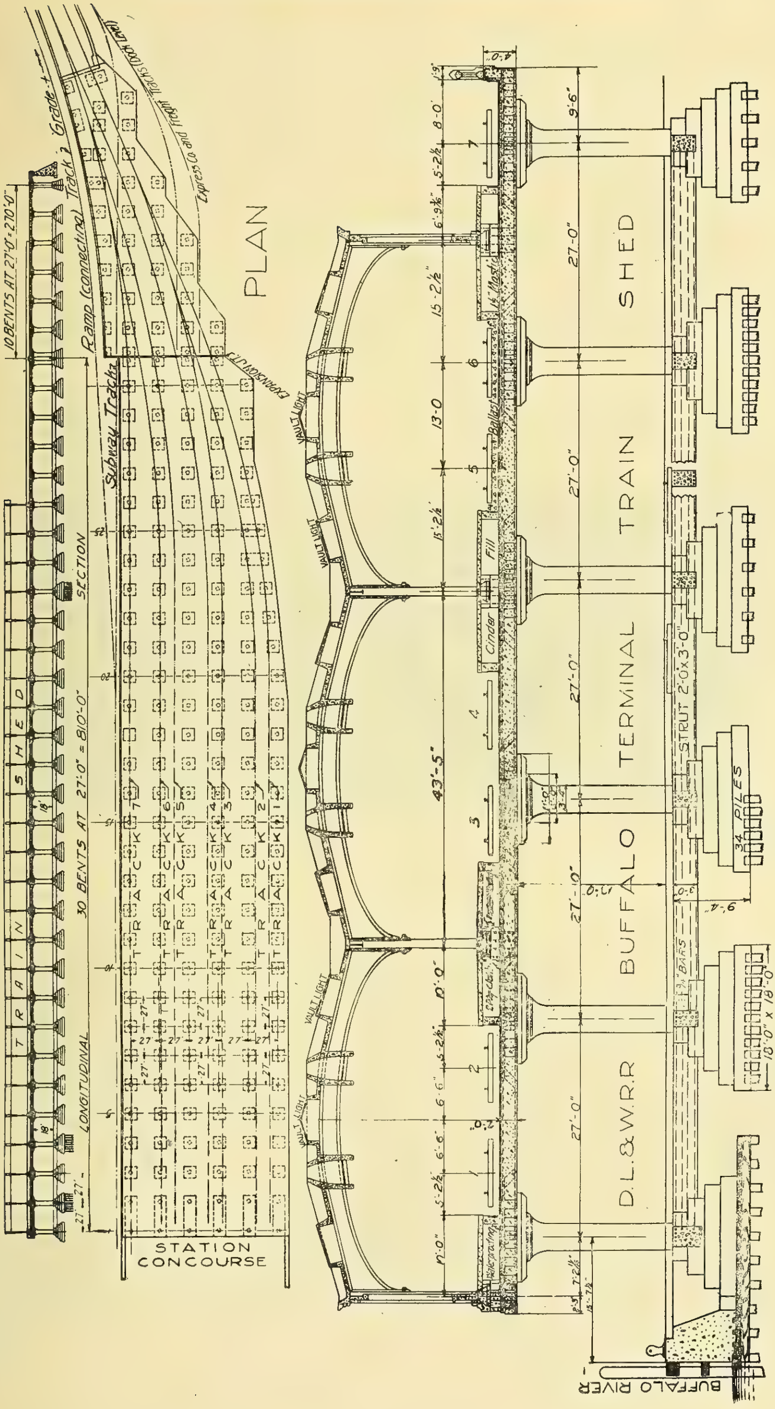

FIG. 3.—DETAILS OF REINFORCED-CONCRETE FLAT-SLAB TRAINSHED OF LACKAWANNA'S BUFFALO TERMINAL.

to about the third point of the panel, ending at the intersection of the center line of track 1 with the side of the drop panel of column in the same bent; another break forms almost normal to this main crack radiating from it and continues parallel with and about four feet off the center line of track 1. This crack extends completely through the center of the large drop panel common to the two outermost columns of bent 25, but like the main crack does not seriously impair the strength of the slab.

A plausible explanation of the cause of the cracking which happened before any live-load was applied might be advanced. This section was poured July 14, 1916. The waterproofing was laid during the following winter and it was on this panel that the kettles for melting the asphalts were placed, resulting in high temperatures in the slab. There is the possibility that the subsequent sudden cooling of the slab in zero weather caused the cracking above described. Sudden atmospheric changes of wide range are common in locality around Buffalo.

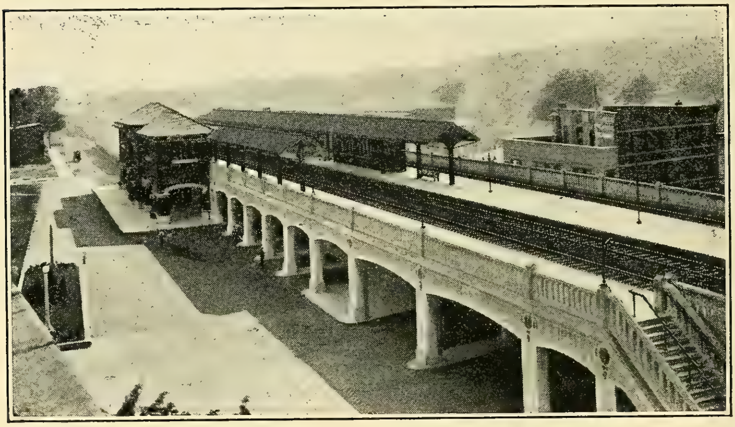

FIG. 4.—FLAT-SLAB BRIDGE CARRYING LACKAWANNA AT SOUTH ORANGE STATION.

This hypothesis is not advanced so much in an endeavor to substantiate the writer's previous remarks—that the constant tensile resistance of the slab under ordinary conditions has the effect of preventing cumulative action of the stresses at any particular section—but is given rather for its value as a warning in anticipation of what may occur to any concrete bridge slab if too much heat is applied locally in extreme cold weather to comparatively thin slab for whatever may be the purpose.

SOUTH ORANGE STATION VIADUCT.

The second application of the flat-slab construction made by the Delaware, Lackawanna and Western Railroad solved in a very acceptable manner one of the perplexing problems encountered in grade crossing elimination through populous sections. The difficulty develops when it becomes necessary to acquire abutting property for the expansion of tracks and station facilities, in which case the property is usually rated at an exorbitant value.

This condition prevailed in connection with track elevation work through South Orange, N. J., where the acquisition of more right of way would have been necessary for an additional third track and island platform together with a new station if the latter were to be built in the usual manner alongside. The necessity of purchase was obviated advantageously by the adoption of a flat slab viaduct, 79 ft. in width and 426 ft. in length, under which the station and all its appurtenances were built within the confines of the original right of way. (See Figs. 4, 5 and 6.)

The proximity of South Orange Ave., a county highway, was an important consideration in favor of the slab construction since the easterly end of the viaduct is carried over this main thoroughfare. Included in the facilities provided under cover of the slab are a concourse connecting the station with the avenue and its trolley line, parking space for vehicles, a baggage platform and a heating plant apart from the station.

In addition to the three tracks, the viaduct carries an eastbound platform 14 ft. 4½ in. in width and a 25 ft. island platform between the middle or express track and the westbound local. The 25-ft. width was fixed by clearance requirements on either side of the shelter houses built on the platform.

There are many advantages in addition to that of economy to be gained by this type of construction. It permits of more effective architectural treatment; because of its shallow floor depth the track can be laid in ballast which is a very important consideration in track construction; there are no girders projecting above the deck to encroach upon the lateral clearance of the motive power or to interfere, as in this case, with the construction of the platforms; the rigidity of the structure is noteworthy since no noticeable vibration is developed with the simultaneous passing of heavy locomotives at high speeds on all three tracks; by reason of this rigidity and of the ballasted floor, the rumbling noises common to structural steel bridges are very much subdued.

CENTRAL AVENUE BRIDGE, ORANGE, N. J.

In continuation with the improvement through South Orange, and following extensive plans to eliminate all grade crossings through its highly developed suburban sections in New Jersey, the Lackawanna Railroad has elevated tracks through the adjoining City of Orange. Here twenty-three more or less dangerous crossings have been eliminated. In this work concrete was used almost exclusively in the construction of bridges.

The same advantages that accrue from the use of flat-slab construction covering large areas prevail for smaller structures. Fig. 7 and Fig. 8 illustrate its application to a small type three-track bridge built on the Orange Improvement over Central Ave., 100 ft. in width. Columns on the curb and along the center line of the driveway divide the deck into eight rectangular panels, two in the width and four in the length of the structure, with dimensions as shown in cross-sections of Fig. 7.

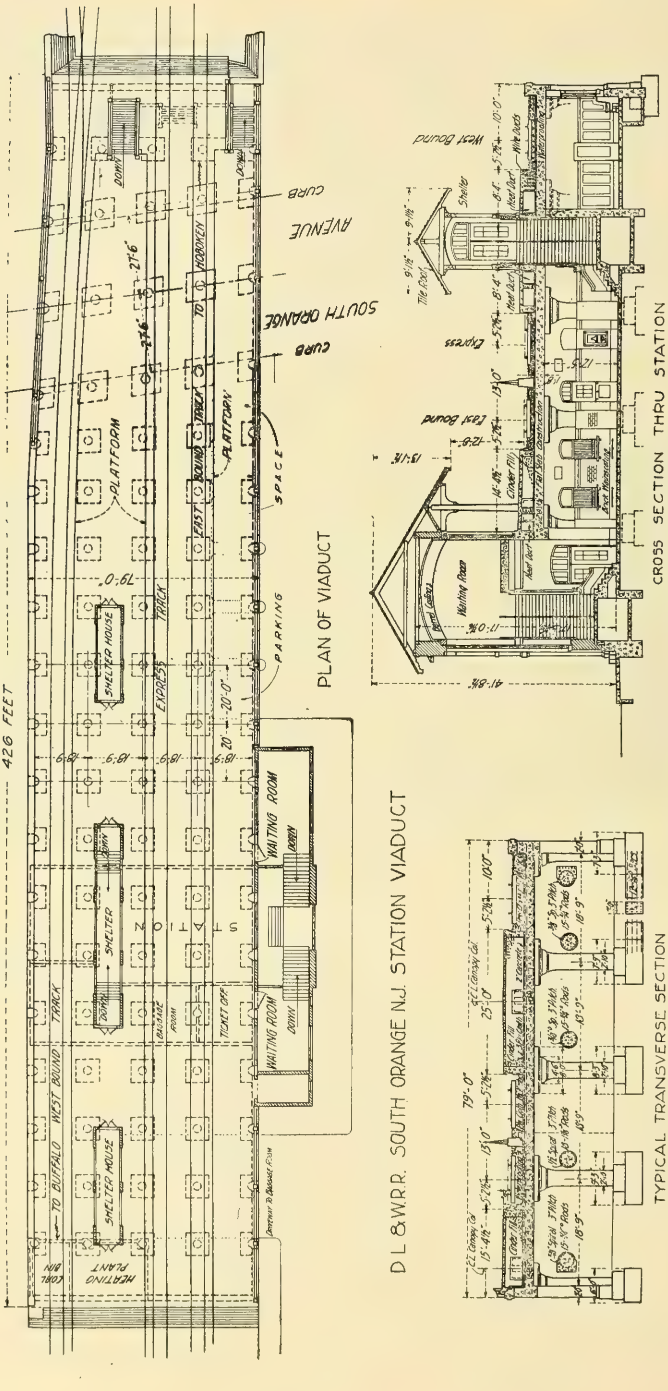

FIG. 5.—DETAILS OF THE SOUTH ORANGE STRUCTURE.

The simplicity of the structural details of the flat slab offers an opportunity to correct the troublesome conditions encountered in bridge abutment construction. The writer calls attention to the abutment development of the Central Avenue Bridge which resulted from the conception of maintaining throughout the deck the positive plate action attributable to the flat slab. This was to be effected by an end column support to replace the somewhat complex action in supporting the slab on the full width of the abutment. The columns are set in recesses built in the abutments as shown in sections A-A and B-B and also in the isometric drawing of the corner column, which shows the sequence of the construction. The slab is cantilevered beyond the abutment, and built integrally with the slab is the suspended beam or apron which is to prevent the back fill and drainage from percolating through the construction joints. The cantilever has a theoretical significance in giving greater balance in resistance to the negative moment over the columns. The abutment is in fact a retaining wall, since it takes no slab reaction, and it was possible to reduce its section for the reason that the suspended apron of the slab reduces to a considerable extent the live- and dead-load surcharge pressure against the back of the wall. The reduction in concrete is a saving over ordinary concrete bridge abutments and an appreciable saving over the massive abutments required in support of structural steel bridges, where the top width is fixed by wide bearing plates, or shoes, and a back wall. The

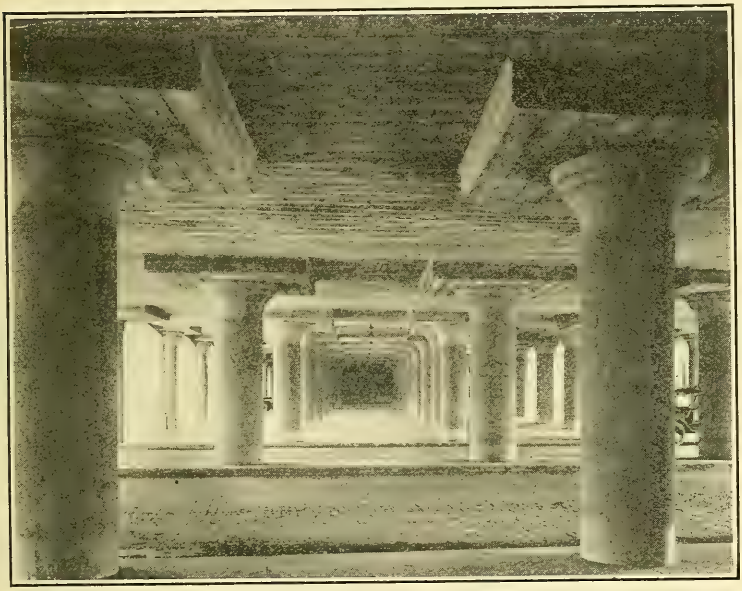

FIG. 6.—UNDERSIDE OF SLAB AT SOUTH ORANGE.

wide bridge seat retains water, snow and ice, which are sources of much trouble resulting from their marked deteriorating action on the steel and concrete. Here, again, it is to be noted that the waterproofing details are reduced to the very simplest arrangement.

Relative to the comparison of the cost of structural steel with the flat slab for the small type railway bridge, it has been found in a number of estimates that a very appreciable difference existed in favor of the flat slab. The statement that the cost of structural steel in one case exceeded the cost of the flat slab by 200 per cent may seem somewhat surprising. This result was obtained where deck construction would have been required on account of a yard layout involving crossovers on the bridge, and a shallow floor depth made necessary because of close vertical clearances. The estimate included the price of structural steel at its high-water mark. The appreciable saving augmented by the present high price of structural steel, which is likely to continue for some time, should give added impulse to the consideration of flat-slab construction.

From the standpoint of appearance and quantities involved a specific comparison of the flat-slab bridge can be made with a flat-top bridge, the deck of which is the common slab of rectangular cross-section reinforced in the one direction for continuous action over a series of piers. Fig. 9 is an example of the latter type spanning Waverly Place in track elevation through Madison, N. J. The spans of this bridge are almost identical with the spans over Central Ave., since here the street is also 100 ft. in width, but the slab reinforced in one direction is 12 in. deeper than the four-way reinforced slab of Central Ave. Bridge, exclusive of the drop panel. If this type were used at Central Ave. at the same unit prices, its cost would exceed that of the present structure by 25 per cent. The noticeable advantage of the Central Ave. Bridge is the clearer vision beyond the bridge obtainable from all angles of approach.

D., L. & W. R. R. BRIDGE, NO. 9.99.

That the flat-slab system has flexible possibilities not obtainable by established bridge construction, a plan (Fig. 10) of a study in the renewal and extension of the D., L. & W. R. R. Bridge, No. 9.99, is submitted for consideration.

It is here proposed to remove the present two-track steel bridge for the reason that the abutments, 29 ft. apart, encroach upon the 41 ft. 6 in. new driveway, which has been substantially paved with granite block on a concrete base. The bridge is to be extended for future track development on the southerly end, including the installation of an industrial track, by extension of the present separate sidings which terminate close to the proposed bridge on either side.

Lateral street intersections fixed the 5-per-cent grade of the street on the southerly side to begin at the face of the present steel bridge, thereby materially encroaching upon the vertical clearance for future expansion of tracks at their present elevation. To overcome this difficulty the flat slab is here tilted in the transverse direction to be approximately parallel with the grade

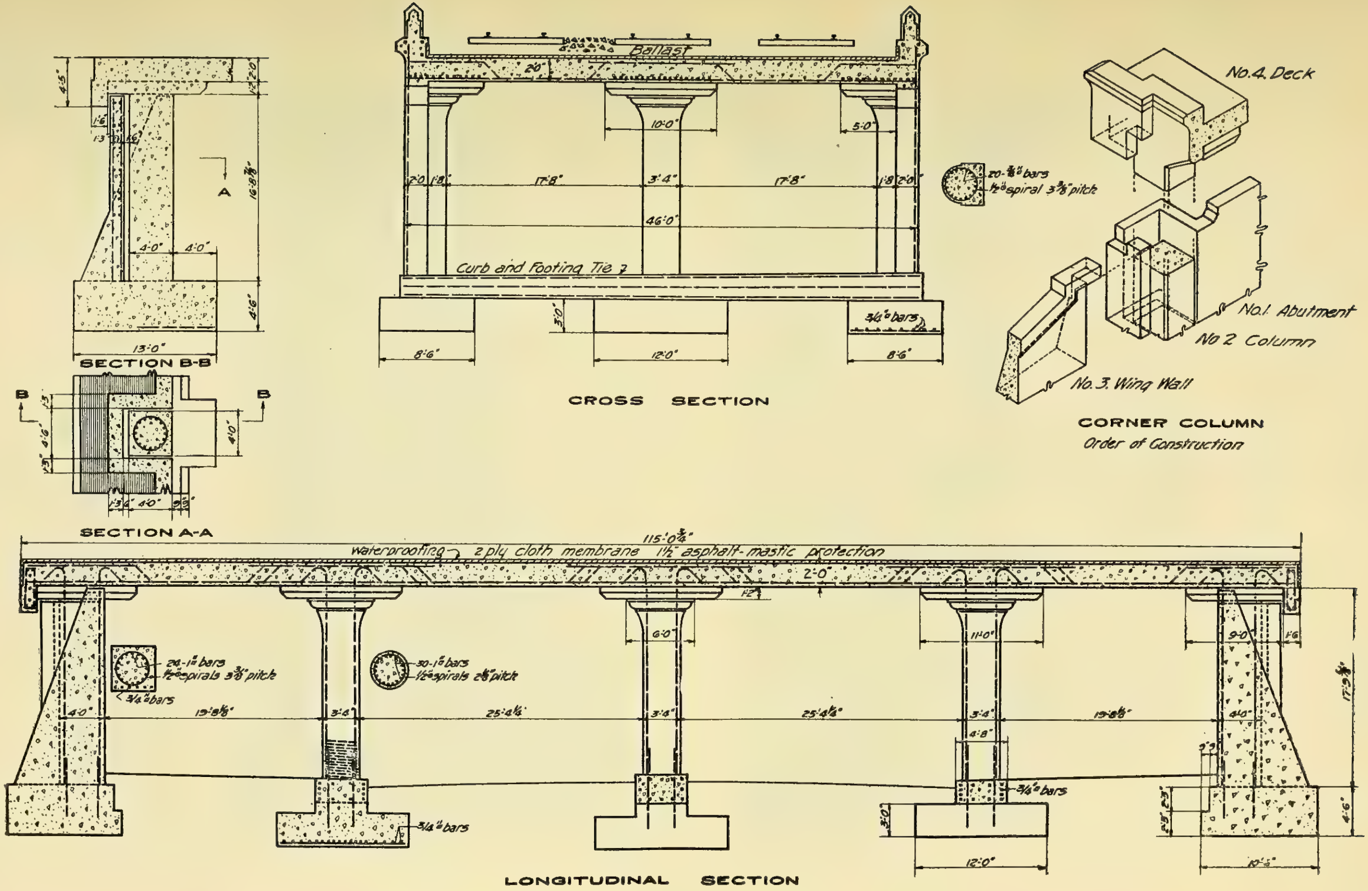

FIG. 7.—DETAILS OF BRIDGE AT CENTRAL AVE., ORANGE, N. J.

of the present crown of street, and in the longitudinal direction parallel with the grade of street, and in the longitudinal direction parallel with the grade of the tracks. This flexibleness and the thin slab have resulted in a very

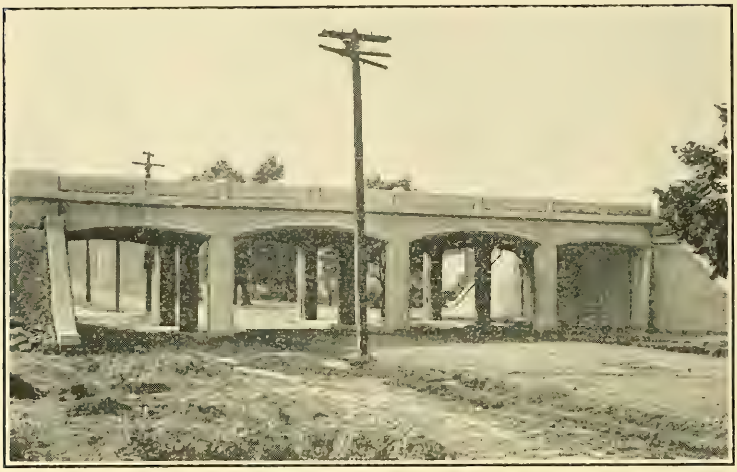

FIG. 8.—VIEW OF THE CENTRAL AVE. BRIDGE, LACKAWANNA R. R.

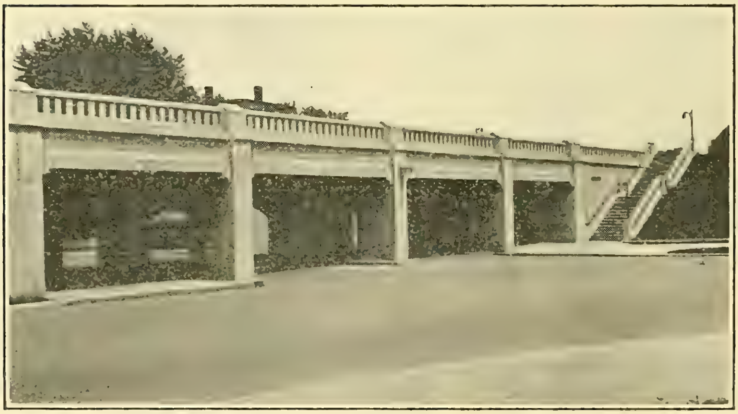

FIG. 9.—WAVERLY PLACE BRIDGE AT MADISON. N. J.

shallow floor depth effecting the preservation of the established well-ballasted main-line tracks, and the extension across the bridge of the industrial siding to the right at its present elevation without resorting to the usual alternative of lowering the street which in this case would be a very expensive operation.

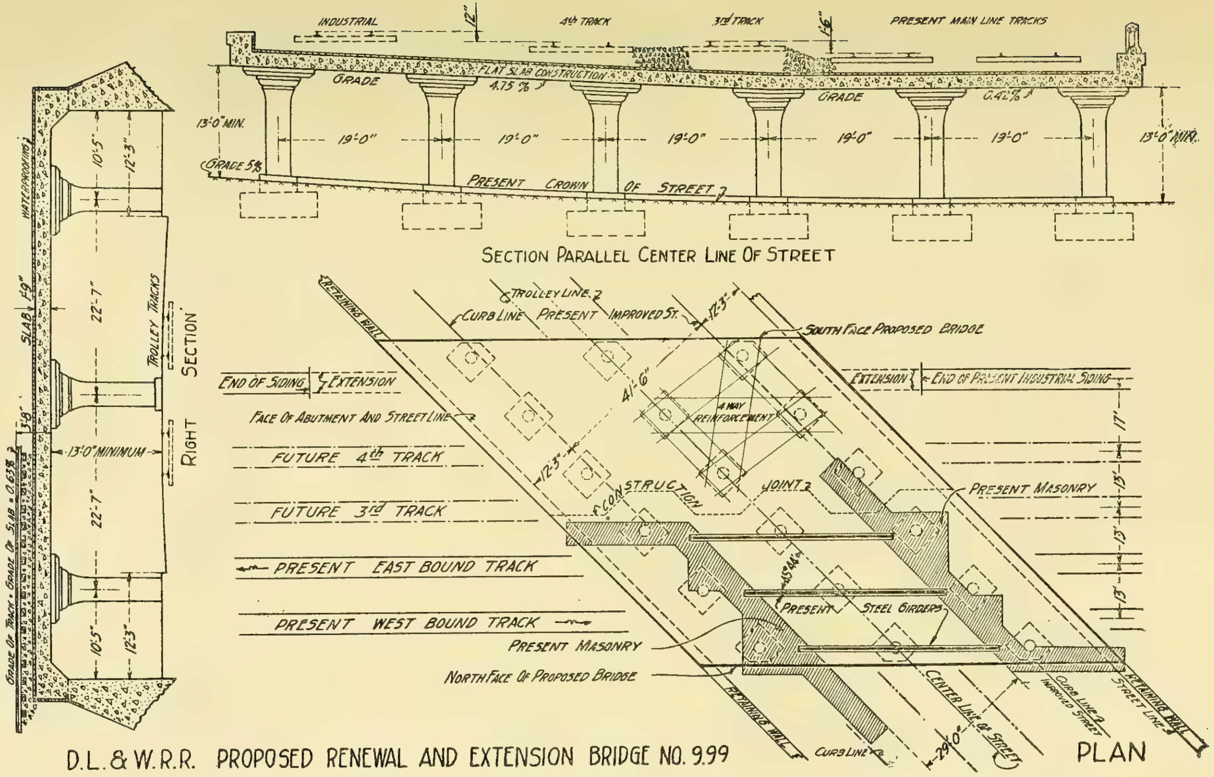

FIG. 10.—A PROPOSED FLAT-SLAB SOLUTION OF DIFFICULT BRIDGE PROBLEM.

When the future third and fourth tracks are laid, they can easily be established at the elevation 1 ft. 6 in. above the present tracks necessary to provide the proper amount of ballast.

The most important consideration in the construction of the small type railway bridge on an established alignment is to maintain traffic without interruption during the operation. This is handled in a number of ways. Where the topography will permit, the alignment is shifted temporarily in order that the bridge might be built clear of traffic, in part or in its entirety. Where the right of way is of limited width and the tracks cannot be shifted, a timber pile bent trestle of 12-ft. spans is driven under traffic and between these bents, after the excavation has been made the abutment and piers only of the new bridge can be built. Long temporary through girders are often used to span out to out of the new abutment lines in order that the entire bridge may be built underneath. If no old girders are available and the only solution is the timber trestle there arises the exclusion of the flat-slab construction, for the reason that the floor system of the new bridge must be erected beyond the bridge site, either in units or in the whole, followed by a quick removal of the trestle stringers and the installation of the completed floor system on the new masonry during hours of least traffic.

This very important consideration of construction is satisfied in the last example by dividing the work into two parts along the construction joint as indicated. This joint is placed without weakening the strength of the slab and so that the southerly half can be built first without interference with traffic and alongside of present structure by removing only a small portion of the old masonry. The main line tracks will be shifted temporarily to the completed half which gives clear field for the removal of the old bridge and completion of the new structure. The bulkheads of the construction joint are arranged to be practically normal to the bands of reinforcement.

The measure of the advantage in cost of the flat-slab railway bridge, compared with other types, varies considerably and is dependent upon the conditions at hand. There seems to be no question concerning the architectural and structural advantages, the latter results in less maintenance and greater permanence. Of immeasurable value is the simplicity of design and the expediency with which the construction can be carried on. These features are emphasized in the last example which, with its 45° 44′ angle of crossing coupled with the grades of the street and tracks, would considerably complicate the details of design and construction of established bridge practice. The flat-slab design will not require any special consideration on account of these complications. Its flexibility offers much opportunity in overcoming and simplifying other inherent complications of the small type railway bridge.

DISCUSSION.

Mr. E. S. Martin.—Mr. Cohen has stated, but not very emphatically, one feature of this flat slab as applied to railroad bridges; that is the very thin thickness of the bridge floor. I think it is about the year 1909 that I was engaged in designing some small bridges of this class for the Soo line in the City of St. Paul. That is the same railroad that owns the terminal in Chicago that is here illustrated. These bridges were street crossings over the railroad and they did not carry the tracks; they did, however, carry interurban tracks. One of these bridges had a span about 28 x 35 ft. between columns, and because of the very low head room, steel ties were used and imbedded in the structural slab, so that on the base of the rail or top of the slab to the bottom of the slab was a distance of about 20 in. Often in elevation work such permissible thin slabs are perhaps more important than the saving in the cost. **Mr. Martin.**

Mr. W. K. Hatt—It is interesting to know that the Soo line terminal in Chicago, of which Mr. Cohen gave pictures, has an elevated slab some 2000 ft. long and 400 ft. wide, erected without expansion joints, which has gone now through several winters without any great damage. Since its erection the lower part has been closed in and used as a cold storage warehouse. The advantage of this is, of course, that in expensive property the ground floor may be used for storehouse purposes and the upper floor for railroad use, and it was said at that time that the rental value of the warehouse extinguished the first cost inside of ten years. This was made out of the elevated slab. **Mr. Hatt.**

ECONOMY IN THE DESIGN OF CONCRETE BUILDINGS.

By Clayton W. Mayers.*

Part 1.

Upon the designing engineer of concrete buildings rests the big responsibility of conservation of building materials. The mere fact that concrete is composed of cement, sand and stone, of which there seems to be an excellent supply, does not in any way relieve the designer of concrete construction of the obligation of careful study of the work in hand in order that no excess of material be used. Most errors made in concrete design are not easily recognized even by experienced estimators of building construction. For instance, hidden away inside a column there may be reinforcing steel which should be elsewhere doing work at less expense to its owner, as would have been the case had the designing engineer given proper thought to the design of this column. A large percentage of the floor space occupied by columns might be storage space for the same reason. The beams may contain an excess of steel reinforcement simply because it was less trouble to call for straight top rods to take care of negative bending than it was to determine where and how bends could have been made in order to have used the least amount of reinforcement in the design. Flat slabs may have a number of individual rods over the top of each column head, where a few more bottom rods should have been bent up to take care of this negative bending. And yet this entire building may have been designed in accordance with the recognized standards of concrete design. These errors are not errors in computations, but are errors of careless design and the result is dire waste of material.

In most cities building plans are O. K.'d by responsible engineers authorized by the city to pass upon these plans before the work is allowed to proceed. Errors in computation are usually detected in this process, but who ever heard of one of these authorized engineers returning a set of plans with suggestions for a more economical design? The design either "gets by" or comes back for correction on account of errors in computation, etc.

Hence, if the designing engineer does not study economy in the design of his work, he may be reasonably sure it will get very little such study from anyone. Designs prepared without this special study are sure to show a waste of building material, and the building is no better, and serves no better purpose, because of this extra expense.

Recognized standards are observed by most designers of concrete buildings as regards stresses and strains, factors of safety, etc., but unfortunately no rules, tables or data are at hand which will solve the problem of maximum economy in the choice of the various methods of concrete construction which may be used. Each building presents new problems. It is a case of careful study in an intelligent way and the designer must do this work well if he would "do his bit" and at the same time keep or build up his reputation.

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Is there any position more humiliating to the designing engineer of a contemplated structure than to have a bidder who is estimating the cost of this work submit to the prospective owners a more economical design based on the same fiber stresses as were used in the original layout? This is not an uncommon event and uncomfortable complications always arise.

A general survey of conditions and inspection of the possible methods of construction usually constitute the first thought given to a new problem of structural design. By this inspection a process of elimination is set up and finally the engineer considers only a few schemes which could be well employed to give the owner a structure suitable for his purposes. The next step usually consists of viewing the several schemes from every angle in order to study their individual merits. Each layout possesses different advantages, some of more value than others, but each one would answer the purpose very well. For instance, a beam and girder type of floor construction may offer advantages in the way of hanging shafting if the building is to be used for certain types of manufacturing. Column spacing would perhaps work out to better advantage in one scheme than in another. Thus the discussion continues with here and there a remark about the probable cost of this and that. A decision is usually made in favor of the scheme offering the most advantages even though they are trivial. The plans are drawn up on this basis and the work proceeds. The detailed design is finished with about the same attention to costs as have been given to the selection of the type of construction used. Generally, the owner of the completed building is satisfied, being ignorant of the fact that he could have had just as good a building for less money.

The average concrete designer makes no claim to being an estimator. In fact, he does not think it is necessary to be an estimator even of the materials with which he works. It is a fact that a large majority of men employed in the design of concrete buildings have hardly any idea of the cost of the work they are laying out, and what is more, they do not know how to find this out for themselves. Surely, if an engineer designed a structural steel girder he could tell with reasonable accuracy what it would cost by computing the weight and getting the market price of the structural steel and the labor cost of erection. Estimating the cost of concrete work is a little more complex, but each step is very similar and the process is the same.

A designer of concrete structures should think continually of costs, but in order to think intelligently of the cost of his work he must know how to calculate approximately the cost of his designs. In no other way is he able to determine which one of his studies will serve his purpose at the least expense.

It should be borne in mind that in making designs for comparative costs, it is not necessary to work to as great a degree of accuracy as for the finished plans. Rough designs, accompanied by rough sketches, will furnish enough information for his study. In case the comparative costs of two schemes should work out the same, a more careful design might become necessary. A little practice on the part of the designer will soon reveal to him to what degree of accuracy he must work in order to get satisfactory results.

The process of estimating these various designs for comparative cost purposes is not nearly as difficult as may be supposed. Concrete is measured by the cubic foot or cubic yard, forms by the surface measurement in square feet and reinforcement by the pound or ton. After the quantities have been calculated for the various designs, unit prices are fixed and the total cost of the member estimated. It is usually here that the engineer throws up his hands. In fact, it is very likely that he knows but little about the prices of this class of material and labor, and in his rush of work he has not kept in touch with the fluctuations, and feels he does not have time to inform himself properly on this subject. Again, it should be understood that it is not necessary to fix absolutely accurate unit costs to these quantities in order to obtain reasonably accurate cost comparisons. As long as the same unit costs are used for similar types of work in the various designs, the comparative costs will be surprisingly accurate. In fact, some of the unit costs may be in error 25 per cent or 30 per cent, and yet the resulting costs will show unquestionably which type of construction should be used. For example, the quantities for two designs (a) and (b) for an interior column are given here and these quantities are priced for current normal conditions, under "Estimate A," and another estimate for the same quantities with the unit prices grossly in error is shown in "Estimate B."

ESTIMATE A.		ESTIMATE B.	
(Scheme a.)		(Scheme a.)	
Conc. (1 : 1½ : 3), 52 cu. ft. at 36½c	$18.98	Conc. (1 : 1½ : 3), 52 cu. ft. at 27c	$14.04
Forms, rd. steel	15.00	Forms, rd. steel	19.00
Reinfct. (vert.), 514 lb. at 5c	25.70	Reinfct 514 lb. at 3¾c	19.28
Sdirals, 264 lb. at 5½c	14.52	Spirals, 264 lb. at 4c	10.56
Lost fl. space, $\frac{7}{10}$ sq. ft. at $2.75	1.92	Lost fl. space, $\frac{7}{10}$ sq. ft. at $3.50	2.45
Total	$76.12	Total	$65.33
(Scheme b.)		(Scheme b.)	
Conc. (1 : 1 : 2), 52 cu. ft. at 43c	$22.36	Conc. (1 : 1 : 2), 52 cu. ft. at 32c	$16.64
Forms, rd. steel	15.00	Forms, rd. steel	19.00
Reinfct., 245 lb. at 5c	12.25	Reinfct., 245 lb. at 3¾c	9.19
Spirals, 264 lb. at 5½c	14.52	Spirals, 264 lb. at 4c	10.56
Lost fl. space, $\frac{7}{10}$ sq. ft. at $2.75	1.92	Lost fl. space, $\frac{7}{10}$ sq. ft. at $3.50	2.45
Total	$66.05	Total	$57.84

It will be seen that although the comparative total costs of the schemes (a) and (b) are entirely different in the two estimates, the resulting comparative costs in both "Estimate A" and "Estimate B" show conclusively the design (b) is the cheaper column to build. It will also be noticed that the percentage of cost saved by using design (b) is about the same in both estimates. However, the alert engineer will soon become as interested in having his unit costs in accordance with current prices of material and labor as he is in having his design correct.

Contrary to the opinion of most engineers, the concrete building design calling for the least amount of material is not always the cheapest building to erect, as such a building may call for much more labor. Form work is a big factor in the cost of concrete buildings, and this phase of the operation must be given careful consideration in order to simplify the construction of the form work as much as possible. Study must be made also to determine whether the complexity of forms in a comparatively light design would not make the final cost of the building in excess of a building designed of simpler yet heavier construction. Concrete floors designed on the flat slab method sometimes have considerably more material in them and yet work out cheaper than a beam and girder type designed for the same conditions. Placing reinforcement costs more per ton and forms more per square foot in a beam and girder construction than the same operations in a flat slab construction. In laying out floors of the beam and girder type, the addition or omission of one beam per bay may influence the cost of the design a great deal. Changes in column spacings will also have the same effect. It is only by making the design of a typical floor bay of the various schemes considered and getting the quantities and costs of these schemes that it will be possible to tell definitely which method should be used. Many times concrete columns should be composed of a richer mix of concrete and have less reinforcement. In a building of several stories it is necessary to devote considerable study to the design of columns in order to locate the point where the mixes should change, where spirally reinforced columns should be introduced, and also to consider carefully the loss or gain of floor space occupied by columns. It will be necessary to make several sketch designs and calculate the cost of each. Thousands of dollars may be wasted by improper column design and still the error is one which would not readily attract attention. There is a certain type of design for every part of the construction which will show maximum economy and it is up to the designing engineer to calculate the costs of his various designs and determine for himself which one should be used.

Up to this point this article has emphasized, principally, the necessity of making several preliminary designs of the various members of a concrete building and calculating the cost of each design before the final layout is begun. Not much light has been shed upon the method of obtaining unit prices to fix to the quantities of material and labor. Unit prices are subject to wide fluctuations. Markets, labor, location of the work in question, speed of the operations, etc., and many other items enter into the making of these costs. However, as stated before, these unit prices need not of necessity be extremely accurate, and the designing engineer need not feel that he cannot price closely enough to obtain fairly accurate results.

A list of approximate unit prices have been tabulated here which may be used to calculate the comparative costs of the principal members in a concrete building. Judicious use of these unit costs will enable the designer to incorporate in his design the most economical methods and at the same time develop a keener eye for economical construction.

Concrete (1 : 2 : 4 mix), per cu. yd.	Cement, $1\frac{2}{3}$ bbls. at $2 per bbl. at the job			$3.33
	Sand, $\frac{1}{2}$ cu. yd. at $1.50 per cu. yd. at the job			.75
	Crushed stone, $1\frac{3}{10}$ tons at $2 per ton at the job			2.60
	Plant, cost per cu. yd.	Freight charges	$0.05	
		Rental of mixer, etc.	.35	
		Purchases	.45	
		Labor	.40	
				1.25
	Labor of mixing and placing			1.25
	Total cost per cu. yd.			$9.18
	Total cost per cu. ft.			.34

Concrete mixed in the proportion of 1 : $1\frac{1}{2}$: 3 will require about one-third of a barrel more cement per cubic yard. This will add about 67 cents to the cost of one yard of concrete in place, making the unit price about $9.85 per cubic yard, or $36\frac{1}{2}$ cents per cubic foot. If a 1 : 1 : 2 mix of concrete is used, the cement will be increased about $1\frac{2}{10}$ bbl. over and above that used in a 1 : 2 : 4 mix. At $2 per bbl. this would make the cost of 1 : 1 : 2 mix concrete about $11.58 per cu. yd., or 43 cents per cu. ft. In large plain concrete footings it is sometimes advisable to use a concrete mixed in the proportion of 1 : $2\frac{1}{2}$: 5. Concrete mixed in this proportion requires about $\frac{3}{10}$ of a barrel less cement than 1 : 2 : 4 mix. Figuring cement at $2 per bbl., concrete mixed in the proportion of 1 : $2\frac{1}{2}$: 5 works out at approximately 32 cents per cu. ft. in place.

In making estimates for the cost of concrete in place, the most uncertain element entering into this cost is the item of "plant." The cost of "plant and tools" varies greatly with different building superintendents, and depends largely upon the foresight of the persons responsible for the layout of the job operations. The number and location of the mixers, towers and runs used on the job, layout and extent of storage space for aggregate, source and expense of power, etc., distance over which concrete machinery has to be transported, good or bad mechanical conditions of rented machinery, rental rates of machinery, replacement of missing shovels and other tools and many other variable expenses go to make up this cost. The size and shape of the building, as well as the speed of the operations, play an important part in this cost. The "plant" cost for a job containing 6,000 cu. yd. of concrete need not necessarily be one-fifth more than a job containing 5,000 cu. yd. of concrete. The "plant" will, of course, cost more for the job containing 6,000 cu. yd. of concrete, but since the cost of erecting and dismantling the "plant" work for both jobs may be the same, the extra cost of "plant" for the larger job will be principally extra depreciation or rental, fuel, power, wear and tear and loss of tools. However, "plant" expense enters into all concrete costs and must be included in the unit price of concrete if we would get a reasonably accurate idea of the ultimate cost of the work. At the present high cost of all building materials and labor, "plant" costs cannot be safely assumed to be less than $1 per cu. yd. and will very seldom run as high as $2 per cu. yd. of concrete. Owing to this wide variation in the cost of "plant," it is necessary in estimating concrete to strike an average cost which, while not accurate, will cover the usual "plant" work, and give a unit cost for concrete in which all items of material and labor have been considered. It is with this in view that a "plant" cost of $1.25 per cu. yd. has been used in making up the unit cost of concrete in place as given in the above tabulation.

In calculating the amount of materials necessary to make 1 cu. yd. of concrete, it will be noticed that the only change made in the quantities for the various mixes has been in the amount of cement used. It has been assumed that a cu. yd. of 1 : 1 : 2 concrete will require the same quantity of sand and crushed stone as a cu. yd. of 1 : 2 : 4 concrete. Theoretically this is not true, but in general practice there is some waste of material and it has been found that the small differences of aggregate used in the various mixes of concrete in a building are negligible. A very large part of the concrete in a building is a 1 : 2 : 4 concrete, therefore, the aggregate quantities of 1 : 2 : 4 mix are generally used for all concrete work and the cement alone is changed for various mixes. It will also be noted that the quantity of cement, sand and stone used here is somewhat in excess of the amount usually given in the tables published in various text-books. It must be borne in mind that the waste of materials on the job must be absorbed and the quantities in tables compiled by laboratory tests must be somewhat increased. It is actually necessary to estimate on about 1⅔ bbl. of cement to make 1 cu. yd. of 1 : 2 : 4 concrete on a job where the usual construction methods are employed and in other mixes of concrete the cement should be proportionately increased.

The prices of concrete work as tabulated here are about 30 per cent in excess of pre-war prices and 50 per cent more than the prices of 1913. These costs based on the present high cost of material and labor should be adjusted from time to time as necessary.

The cost of steel reinforcement is extremely erratic in its fluctuation, but at present it may be assumed at $90 per ton exclusive of the labor of bending and placing. It will cost from $6 to $15 per ton to cut, bend and place this reinforcement, $100 per ton, or 5 cents per lb., being a unit price which may be used to give reasonably close cost ratios. Reinforcement requiring much bending and made up of small bars should be figured about ½ cent per lb. higher than steel requiring only a small amount of bending. Spiral reinforcement for columns should be figured at an extra cost of about ½ cent per lb. over and above plain bars. In estimating the weight of spiral reinforcement it should be remembered that about 7 per cent should be added to the weight of the spirals for welding laps. Also, it will be necessary to add about 3 lb. per lin. ft. of column for spacers used to hold the spirals in proper pitch.

Forms for round columns are usually made from sheet metal and in flat slab construction it usually works out cheaper to use round interior columns formed with this material, However, the cost of forming an interior column 26 in. in diameter for flat slab construction is about the same as forming a column 20 in. in diameter designed for the same purpose. This being the case, it is not necessary to consider the difference in the cost of forms due to different diameters of round interior columns. It may be well to remember that it costs somewhat less to build an interior column having a head by using a steel form than it does to form the column of wood, as the cost of forming the head in wood is no small part of the column cost. The list of unit prices given here covers the cost of labor and material for form work for the principal operations in a concrete building, but are tabulated for use in making comparative estimates only. It must be borne in mind that these unit prices are for the use of the engineer in weeding out the more expensive designs and are not to be used for making actual estimates of buildings without regard to conditions and what not. While these costs might be more or less useful in arriving at the total cost of a concrete building it should be remembered that they are only approximate units to be used for the purpose outlined herein.

TYPE OF CONSTRUCTION.	SQ. FT. COST. (Surface Measurement.)
Forms for flat slabs, including drop panels	$0.09
Slab, beam and girder construction, slabs to span not less than 9 ft	.12
Slab, beam and girder construction, slabs to span not less than 7 ft	.13
Slab, beam and girder construction, slabs to span not less than 5 ft	.14
Column forms	.15
Floor beams and girders, not including slabs	.16
Wall beams	.14
Partitions and wall forms	.15
Footing and foundation forms	.15
Round steel column forms, including heads, each	15.00

Now that the methods of arriving at the comparative costs of the various types of concrete construction have been outlined, it is believed the designer will be able to work more intelligently regarding the cost his work involves. Typical dimensioned sketch cross-sections of the building from the roof slab to the footings should be made and the work of estimating done from these sketches. In this way the extra column lengths required to obtain the same clear story heights will enter into the estimate. This is quite a factor in comparing flat slab with beam and girder designs. Estimates made from these cross-sections for a length of building equal to one bay only, is the usual practice. In this way the cost per lineal foot of building as well as the cost per square foot of floor space may be calculated. Comparisons of costs made in this manner are genuine proofs to the designer that he is giving the design proper study for economy, and will result in a conservation of building materials, save good dollars for the owner, and establish for the engineer the reputation of being a designer of economical concrete buildings.

Part 2.

Probably no part of a concrete building is simpler to design than the columns, and because of this simplicity the designer is very likely to give this part of his computations very little special study. It is also true that no part of a concrete building can conceal so effectively the lack of economical design, as can the columns.

The economical design of the columns for a concrete building of only one or even two stories in height, is not a matter requiring much special study, but in buildings several stories in height the subject is one of vast importance. It is not possible to design columns showing maximum economy without careful consideration of several important facts. Engineers designing concrete buildings realize that a richer mix of concrete costs more than a leaner mix. They realize that to offset this extra cost of a richer mix of concrete in column design, there is a corresponding decrease in reinforcement which results in a change in the total costs of the concrete columns. The manipulation of these mixes of concrete in order to determine the most economical column construction is a subject for real study and to accomplish this end the engineer will find it necessary to make several trial designs and calculate the cost of each design. For example, a column 26 in. in diameter, composed of concrete mixed in the proportion of 1 : 1½ : 3, reinforced with eleven 1-in. round rods and 1 per cent spiral hooping, will carry about the same load as a column of the same diameter composed of 1 : 1 : 2 concrete, reinforced with seven ⅞-in. round rods and 1 per cent spiral hooping. As both of these are good designs, the question arises as to which one would be the most economical. Assuming the unit price of 1 : 1½ : 3 and 1 : 1 : 2 concrete at 36 cents and 43 cents per cu. ft. respectively, vertical reinforcement at 5 cents per lb., and spiral hooping at 5½ cents per lb. in place, it can be clearly shown that the column composed of 1 : 1 : 2 concrete will prove to be the more economical one to use. The point at which the column mixes change and where spirally reinforced columns should be used is determined only by making these comparative estimates.

It is not uncommon to see detailed plans calling for a lap in all the rods in a lower story column without regard to the fact that the column above may call for a lesser number of rods for its reinforcement, and it is only necessary to lap part of them. This is real waste and shows careless design which will run into money faster than the designer suspects. For illustration, the first story wall columns of a concrete building are 36 x 34 in., reinforced with twenty 1⅛-in. round rods. The second story wall columns are 36 x 30 in., reinforced with fourteen 1⅛-in. round rods. A lap of thirty diameters is called for in all column rods. If the entire twenty rods in a first story column are lapped into the second story column it means that six of these twenty 1⅛-in. rods have been unnecessarily lapped and consequently this extra reinforcement wasted. Had only fourteen of these first story column rods been lapped into the second story column instead of twenty, a saving of about 17 lin. ft. of 1⅛ in. round steel rod would have been made. This reinforcement, figured at 5 cents per lb., would have shown a saving of about $3 at this one point. Multiply this saving by the number of columns in the building where such laps occur and it becomes no small item. The expense of a case of this kind becomes considerably greater when the column in question is a wall column on which is superimposed a so-called turned up wall beam designed to be poured with the floor slab. In this case the specified lap begins at the top of the wall beam instead of the top of the floor slab and extends upward. For example, suppose a wall beam, extending 14 in. above the second floor, designed to be poured with the slab, had been superimposed on the 36 x 34-in. wall column just discussed. In this case the lap must be measured from the point where pouring is stopped. If the twenty $1\frac{1}{8}$-in. round rods are all carried up into the second story column for a lap of 30 diameters it means that all the rods must extend to a point 4 ft. above the second floor, when in reality it is necessary to carry only fourteen of these rods to this point, starting the fourteen $1\frac{1}{8}$-in. round rods in the second story column at a point 14 in. above the second floor and extending upward. The loss incurred by carrying the entire twenty $1\frac{1}{8}$-in. round rods into the second story column is about 24 ft. of $1\frac{1}{8}$-in. round rod, which at 5 cents per lb. is about $4, as against the loss of $3 when no wall beam is designed to be poured with the floor slab. Wastes of this nature at the present high price of reinforcement are serious.

In the design of wall columns it will be necessary, usually, to consider the amount of sash and curtain wall required to fill the space between columns, as the smaller the width of the exterior columns the more sash and curtain wall will be required to fill in the space between these columns. This may seem trivial, but it will oftentimes give false impressions of economy if all these seemingly trivial details are not given a place in the estimated comparative costs of the various designs.

Illustrations of the methods of determining the economical interior column are given below. It may be well to add that no attempt is made to consider any of the various methods of concrete design from an engineering standpoint. This paper is not intended to be a text on design in any form, but rather it is intended to present to the designing engineer a method by which he can solve for himself the question of economy in his work. Hence, the reader should study the examples given here with a view of applying the methods of cost calculation to his work and not draw engineering conclusions from any of these illustrative costs.

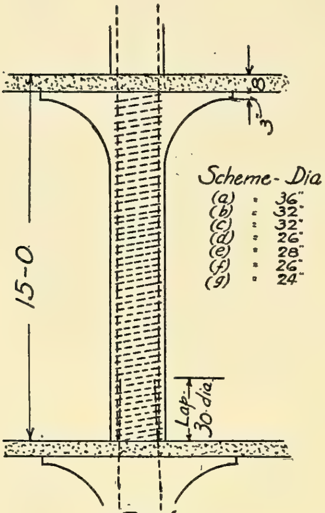
Several comparative designs for any interior column (Fig. 1) are shown here. The comparative costs of the various schemes are worked out in detail, using unit prices principally from tabulations in Part 1 of this paper.

	DESIGN.	COMPARATIVE ESTIMATES.		
Scheme (a)	36-in. dia. col. 11 $1\frac{1}{8}$-in. rd. vert. rods $\frac{3}{8}$-in. rd. bands 12 in. o/c Mix 1 : 2 : 4	Conc.	99 cu. ft. at 34c	$33.66
		Forms	Round steel	15.00
		Reinfct	716 lb. at 5c	35.80
		Lost fl. space	5 sq. ft. at $2.75	13.75
		Total		$98.21
Scheme (b)	32-in. dia. col. 23 $1\frac{1}{8}$-in. rd. vert. rods $\frac{3}{8}$-in. rd. bands 12 in. o/c Mix 1 : 2 : 4	Conc	79 cu. ft. at 34c	$26.86
		Forms	Round steel	15.00
		Reinfct	1,437 lb. at 5c	71.83
		Lost fl. space	$3\frac{1}{10}$ sq. ft. at $2.75	8.54
		Total		$122.25

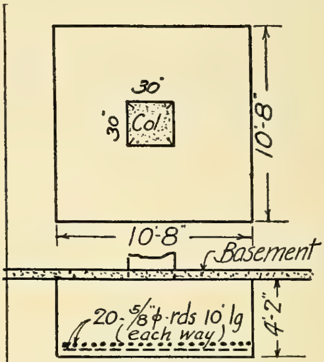
	DESIGN.	COMPARATIVE ESTIMATES.		
Scheme (c)	32-in. dia. col.	Conc	79 cu. ft. at 36½c	$28.84
	12 1⅛-in. rd. vert. rods	Forms	Round steel	15.00
	⅜-in. rd. bands 12 in. o/c	Reinfct	770 lb. at 5c	38.50
	Mix 1 : 1½ : 3	Lost fl. space	3⅒ sq. ft. at $2.75	8.53
		Total		$90.87
Scheme (d)	26-in. dia. col.	Conc	52 cu. ft. at 36½c	$18.98
	11 1-in. rd. vert. rods	Forms	Round steel	15.00
	1 per cent spirals (18½ lb.) per lin. ft.	Reinfct. (vert.)	514 lb. at 5c	25.70
	Mix 1 : 1½ : 3	Spirals	264 lb. at 5½c	14.52
		Lost fl. space	7/10 sq. ft. at $2.75	1.92
		Total		$76.12
Scheme (e)	28-in. dia. col.	Conc	60½ cu. ft. at 43c	$26.02
	20 1⅛-in. rd. vert. rods	Fcrms	Round steel	15.00
	⅜-in. rd. bands 12 in. o/c	Reinfct	1,255 lb. at 5c	62.75
	Mix 1 : 1 : 2	Lost fl. space	1.45 sq. ft. at $2.75	3.99
		Total		$107.76
Scheme (f)	26-in. dia. col.	Conc	52 cu. ft. at 43c	$22.36
	7 ⅞-in. rd. vert. rods	Forms	Round steel	15.00
	1 per cent spirals (18½ lb.) per lin. ft.	Reinfct	245 lb. at 5c	12.25
	Mix 1 : 1 : 2	Spirals	264 lb. at 5½c	14.52
		Lost fl. space	7/10 sq. ft. at $2.75	1.92
		Total		$66.05
Scheme (g)	24-in. dia. col.	Conc	44½ cu. ft. at 43c	$19.14
	10 1⅛-in. rd. vert. rods	Forms	Round steel	15.00
	1 per cent spirals (16 lb.) per lin. ft.	Reinfct. (vert.)	606 lb. at 5c	30.30
	Mix 1 : 1 : 2	Spirals	229 lb. at 5½c	12.60
		Total		$77.04

From the above estimated comparative costs, perhaps the most noticeable fact is that the columns using the 1 : 2 : 4 mix of concrete are among the most expensive. Using this lean mix necessarily produces a column larger in diameter which means, also, a loss of valuable floor space. It will also be noticed that the smallest column designed is not the most economical. The column which shows the most economy in this case is one having a 1 : 1 : 2 mix and about 1 per cent of vertical reinforcement together with 1 per cent of spiral reinforcement. Hence, a rich mix of concrete and comparatively small percentages of steel reinforcement seem to show the most economical results for a column carrying a fairly heavy load.

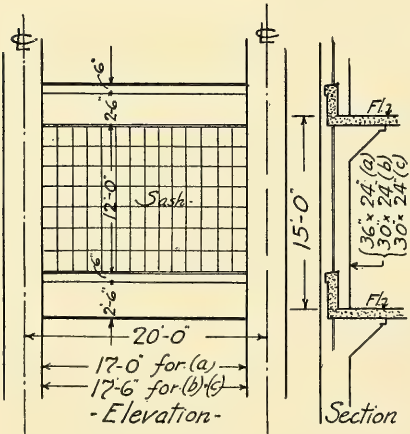
For comparative purposes, the difference in the amount of concrete in the column heads may be neglected as the top diameter of the head usually remains the same throughout the building. The cost of forming the column and its head has been estimated here at $15 each. This is done for convenience in arriving at a total cost of the column shaft. Ordinarily this cost is neglected in making comparative estimates of interior round columns, as it costs about the same to form a round column of small diameter as it does a column of larger diameter. Many other schemes may be designed for this particular column and the comparative costs estimated. However, these several examples, some of which are obviously too expensive to consider, will suffice to give the reader a working knowledge of the methods of calculation employed



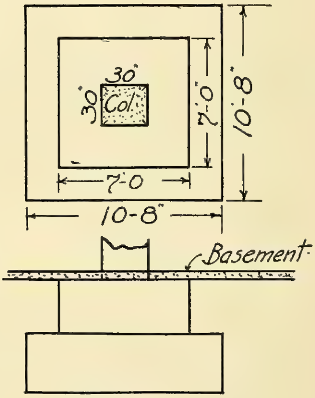
- Fig 1 -



- Fig.3 -
(Scheme - a.)



- Fig 2 -



- Fig. 4 -
(Scheme - b.)

to determine the costs of the various types of interior columns. It is readily appreciated that even though a larger column were somewhat cheaper to build, the additional floor space occupied by this larger column might be worth more to the owner of the building than he would save in the construction of the column. Hence, it becomes necessary to consider the value of this additional floor space as a part of the cost of this larger column. It is difficult to say just what this floor space is really worth. However, a satisfactory way to deal with the situation is to consider the smallest column designed as a basis to which the other columns are to be compared. In the above illustration this column is 24 in. in diameter. Consider the area of floor space occupied by a column equal to the square of the diameter of the column. The additional area occupied by any one of these larger columns is equal to the difference of the square of the diameter of the column in question and the square of the diameter of the smallest column designed. This additional or lost floor area is priced at a unit cost equal to the approximate unit cost per square foot of floor space of the completed building including heating, lighting, sprinklers, etc. The unit cost per square foot of building is calculated by dividing the approximate total cost of the building by the number of square feet of floor space in the building, measurements to be taken "out to out" of the floor plan. For example, a building 200 x 60 ft. and five stories high may cost $165,000 complete. This works out at $2.75 per sq. ft. and for general purposes this will give fairly accurate results for the purpose described above.

In the comparative estimates of the interior column (Fig. 1) given, if we strike out of each estimate the cost of lost floor space, the relative cost of each column will remain unchanged. This is not always the case, and even in our examples it will be noticed that the columns having the leaner mixes show up much more favorably when this item of cost is excluded from the total cost of the column. Frequently, the omission of this item will result in a transposition of the economic order of the various designs. In many buildings the loss of a few feet of floor space is immaterial, but in other cases it is of great importance, as in storehouses or in buildings where the machinery layout would be interfered with by a larger column. Where loft buildings or offices are rented by the square foot of net area the cost of this floor space should be figured at a considerably higher figure than the one given in our tables.

In determining the economical wall column, the method is very similar to that used for interior columns except that the item of the cost of wood forms enters into the estimate. It will be necessary also in designing exterior columns to consider the width carefully, as every inch added or deducted to the width of the column will change the corresponding dimension of wall sash a like amount.

Consideration is given below to the economical design of a typical wall column (Fig. 2) for a concrete building having these columns spaced 20 ft. on centers. For lack of space only three designs will be compared here, but the principles are clearly illustrated and further designs should be treated in a like manner.

Scheme (a)	36 x 24 in. 12 ⅛-in. rd. rods 17 ft. 6 in. ⅜-in. rd. bands 12 in. o/c	Conc.	86 cu. ft. at 34c.	$29.24
		Forms	143 sq. ft. at 15c.	21.45
		Reinfct	777 lb. at 5c.	38.85
		Curtain walls	31 sq. ft. at 75c.	23.25
		Window sill	17 lin. ft. at 60c.	10.20
		Sash and glass	204 sq. ft. at 45c.	91.80
		Total		$214.79

Scheme (b)	30 x 24 in. 10 1⅛-in. rd. rods 17 ft. 10 in. ⅜-in. bands 12 in. o/c Mix 1 : 1½ : 3	Conc.	71⅔ cu. ft. at 36½c.	$26.16
		Forms	129 sq. ft. at 15c.	19.35
		Reinfct	651 lb. at 5c.	32.55
		Curtain walls	32 sq. ft. at 75c.	24.00
		Window sills	17½ lin. ft. at 60c.	10.50
		Sash and glass	210 sq. ft. at 45c.	94.50
		Total		$207.06

Scheme (c)	30 x 22 in. 12 ⅞-in. rd. rods 17 ft. 2 in. ⅜-in. rd. bands 12 in. o/c Mix 1 : 1 : 2	Conc.	66 cu. ft. at 43c.	$28.38
		Forms	124 sq. ft. at 15c.	18.60
		Reinfct	460 lb. at 5c.	23.00
		Curtain wall	32 sq. ft. at 75c.	24.00
		Window sills	17½ lin. ft. at 60c.	10.50
		Sash and glass	210 sq. ft. at 45c.	94.50
		Total		$198.98

The cost of each wall column design includes the cost of sash and glass together with the curtain wall necessary to fill in one bay. For convenience in making these estimates, it is assumed the glass is factory ribbed glass costing 20 cents per sq. ft., including glazing. Steel sash is estimated here at 25 cents per sq. ft., erected and pointed, making a total of 45 cents per sq. ft. for the sash and glass in place. The curtain wall below the sash is figured here at 75 cents per sq. ft. In making the sketches of the exterior wall bay for estimate purposes, no care has been exercised to select stock sizes of steel wall sash. In actual practice, however, this is usually of prime importance. The cost of the extra floor space occupied by the larger wall column has not been considered here as its influence on these particular columns would be negligible.

In the design of concrete footings it often happens that it is difficult to decide offhand whether a plain or reinforced concrete footing should be used. A design of each type of footing should be made and the comparative costs calculated. The engineer knowing the kind of soil these footings will rest upon should price the excavation required at a proper figure. This is a very important part of the footing cost, in fact, many times the most vital part of the estimate for foundation work. In the absence of any more reliable information the unit costs of excavation per cubic yard (not over 5 ft. deep) may be assumed as follows:

Loam or other easy excavation	$0.75 cu. yd.
Gravelly earth containing small stones	$1.00–$1.50 " "
Frozen earth	2.25– 2.50 " "
Rock or ledge excavation	3.50– 4.00 " "
Backfill	.30– .50 " "
Sheeting around excavated holes for footings	.10 sq. ft.

For excavation work over 5 ft. deep and down to 10 ft. deep, the unit cost on the yardage below the 5 ft. depth should be increased approximately 50 per cent. An example is given below with comparative costs for the two types of footings, reinforced and plain, shown in Fig. 3 and Fig. 4, respectively. The excavation is assumed as costing $1 per cu. yd. to remove, and the excavated holes are sheeted close in order to do away with form work around the large footing block.

Scheme (a) Reinforced type (mix 1 : 2 : 4)	Conc.	460 cu. ft. at 34c.	$156.40
	Forms (none).		
	Reinforcement	420 lb. at 5c	21.00
	Excavation	19¼ cu. yd. at $1.00	19.25
	Backfill and level	19¼ cu. yd. at 30c.	5.78
	3-in. sheeting (close)	182 sq. ft. at 10c.	18.20
	Total		$220.63

Scheme (b) Plain type	Concrete 1 : 2½ : 5	507 cu. ft. at 32c.	$162.24
	Forms (top block)	84 sq. ft. at 15c.	12.60
	Excavation	24 cu. yd. at $1.00	24.00
	Excavation below 5 ft. mark	5½ cu. yd. at $1.50	8.25
	Backfill and level	29½ cu. yd. at 30c.	8.85
	3-in. sheeting (close)	270 sq. ft. at 10c.	27.00
	Total		$242.94

As above shown, the reinforced footing is the most economical to use in this case. However, provided stones or "plums" were obtainable at a small expense, the cost of the plain footing could be considerably reduced. It will be noted in the estimates for these two footings that the excavation for the plain footing is the determining factor in its cost. The materials used in the plain footing cost somewhat less than those used in the reinforced type, but the extra depth of the excavation makes the plain type the more expensive one to use. This extra cost becomes still greater when the unit cost of digging increases. In case the reinforced type of footing is built with a sloping top, and a wood form is used for this top, the cost would be about the same as though the concrete were poured up to a level with the top of the footing, and the form work omitted, as above estimated. In some operations the top part of a footing is sloped and the concrete poured "dry." This necessitates a change in the batch, slows up operations and many times does not work out economically. For estimating comparative costs of footings it is not a safe procedure to assume that the top part of the footing will be poured "dry" in order to do away with forms on the slope. Either estimate a form for this sloping surface or figure on the concrete as being poured up to a level with the top of the footing.

It has been previously stated that in the design of the beam and girder type floor, the omission or addition of one intermediate beam per bay may influence the cost materially. Although this problem is usually handled economically by engineers designing concrete buildings which have usual floor loadings and column spacings, it sometimes happens that when unusual floor loadings and column spacings are required, it is necessary for the engineer to determine a layout which will show the most economy. In a proposition of this kind it is first necessary to make the design which looks most likely to be the economical one. Then, two more designs should be made, one having one more intermediate beam and the other having one less intermediate beam. Sometimes the girders should be run in other ways and designs made on layouts entirely dissimilar. Cost comparisons made of these designs will show conclusively which system should be adopted.

For the purpose of illustrating the methods of estimating beam and girder floors with a view to economy, the two schemes shown in Fig. 5 and Fig. 6, designed for the same column spacings and live loads, are estimated here in a comparative way. Only these two layouts are compared here, but other layouts should be estimated in a similar manner, bearing in mind that the more beams and girders in the floor the more expensive the form work becomes.

In scaling the quantities for the comparative estimates of these two designs, it will be necessary to include all the concrete forms and steel reinforcement in one 18-ft. bay for the full width of the building, which is about 67 ft. 6 in. In Fig. 5 the quantities will include the slab over one complete bay, 7 intermediate beams, 2 wall beams, and 4 girders. In Fig. 6, the corresponding quantities will include the slab over one complete bay, 11 intermediate beams, 2 wall beams and 4 girders. Below under the head of "Estimate, Fig. 5" and "Estimate, Fig. 6" will be found these respective quantities to which unit prices have been fixed (a list of which will be found in Part 1), and the total comparative cost of one bay for each scheme estimated.

ESTIMATE, FIG. 5.

Concrete, 825 cu. ft. at 34c	$280.50
Forms, 1,860 sq. ft. at 13c	241.80
Reinfct., 7,300 lb. at 5c	365.00
Total	$887.30

(Unit cost, 73 cents sq. ft. of floor.)

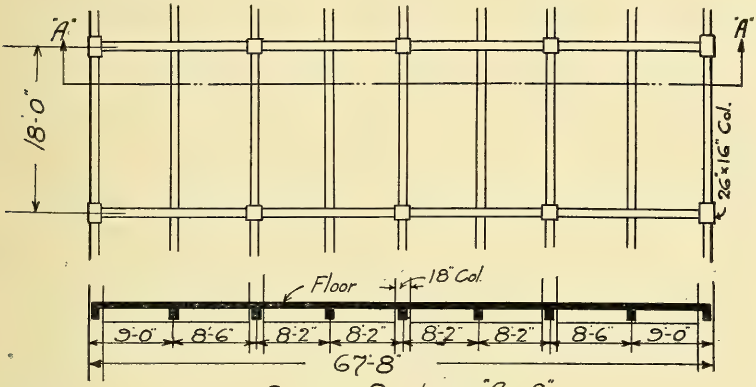
ESTIMATE, FIG. 6.

Concrete, 700 cu. ft. at 34c	$238.00
Forms, 2,000 sq. ft. at 14c	280.00
Reinfct., 6,300 lb. at 5c	315.00
Total	$833.00

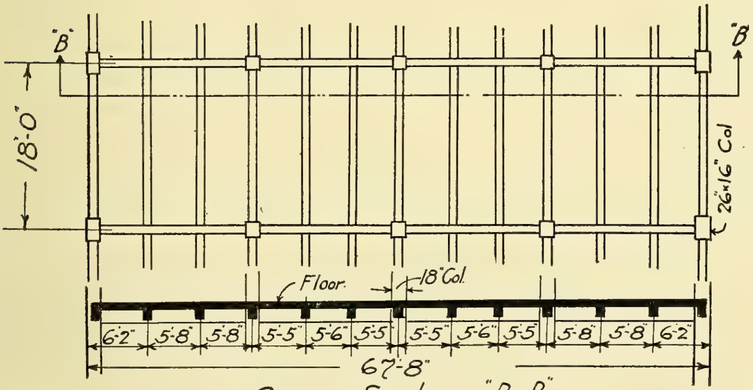
(Unit cost, 68½ cents sq. ft. of floor.)

In "scaling off" the quantities for comparative estimates of beam and girder type floors, care must be taken to carefully consider the laps in the reinforcement. All steel reinforcement actually occurring in the slab and beams should be estimated. In taking off the quantities, also, it will be found most convenient to first get the quantity of concrete, then the square feet of forms, and lastly the pounds of reinforcement. The order of scaling for the form work and reinforcement should be the same as that followed in getting the quantity of concrete, that is, if beams follow slabs in the concrete scaling, beam steel should follow slab steel in the reinforcement scaling. This method will eliminate to a large extent the liability of error and also lessen the work of scaling dimensions since the form areas may be taken directly from the scaled dimensions of the concrete work.

The slight changes in column and footing design which might actually occur in two buildings designed with floors like those above estimated, have



Cross Section "A-A"
- FIG 5



Cross Section "B-B"
FIG. 6

not been considered here as the details of column and footing costs are treated elsewhere. However, in buildings several stories in height this phase of the design should be carefully considered in conjunction with the cost of floor designs when the cost comparisons are made. Even though the spacing of columns remains the same for all schemes considered, the different dead loads may influence the cost of the columns and footings considerably and the different girder depths may make it possible to vary the over-all height of the columns in order to get the same clear head room.

Flat slab floor construction is fast replacing the beam and girder type of floor and, generally speaking, has advantages in appearance and economy. However, there will be places where the beam and girder system will show a lower cost. Where panels between columns are square or nearly so the flat slab usually works to advantage. When columns are spaced unequally or irregularly it is often more economical to resort to the beam and girder type of floor. If the column spacings may be laid out with economy in view, the square bay and the flat slab will generally be selected. However, this selection should not always be made without a proper check by comparative cost estimates. Assuming, for instance, that a concrete storage building is required the width of which may be anywhere from 55 to 65 ft. and sufficient in length to give a certain specified area of floor space. The design is to be a flat slab system and the building is to be built as economically as possible. The engineer will usually make a design for a flat slab system with the columns spaced at distances he believes will show economical results. He should now make two more flat slab designs with the column spacings one foot more and one foot less respectively. Comparative costs made on these three designs will show him the economical standing of the various spacings for the specified live load, and if it does not show him definitely which spacing to use it will give him the hint as to which extreme of column spacings he must still continue to design. It will be necessary to make typical cross-section designs showing the column spacings considered and then calculate the comparative costs of each design for a length of building equal to one bay. It is a simple matter to calculate the number of bays necessary for each type of cross-section to deliver the required amount of building floor space. This being done and the cost of one bay of each type of building being already calculated, the total approximate cost of each type of building is easily found. Adding to these respective estimates the cost of closing in the two extreme ends of the building, the engineer has a very good idea of the comparative costs of the designs he has made.

The principal elements going to make up the cost of a concrete building have been briefly touched upon, with a few illustrations of the methods of making cost comparisons. It is believed the designer will now be able to cope with special problems in a smaller manner, always trying to make actual cost comparisons bear out his decisions whenever it is possible to do so.

The locality of the job in hand plays an important part in the design and cost of the work. Cost comparisons should not be made without regard to this special consideration. Low-priced gravel may take the place of sand and crushed stone. Cement may be unusually cheap owing to very low freight rates. Sand may be procured from a sand bank nearby or possibly from the footing excavation. Crushed stone may be had from a crushing plant close to the site of the job or it may be excessively high in cost. Reinforcement costs may be extremely high on account of transportation. Any one or all of these conditions would influence all concrete units. These local conditions and prices should be studied by the engineer before starting his design. If he does not do so his cost comparisons may not, after all, lead to the correct conclusions.

It was found in making comparative estimates of two systems of floor construction, that when the quantities were priced at the rates prevailing in Buffalo the costs of the two designs were about the same. When New York City prices were applied to the same designs a saving of about 8 per cent of the cost was shown in favor of the design calling for the least concrete and the most reinforcement.

In this paper many designs might have been made and the comparative cost of each design calculated. Building layouts might have been discussed at length with a view to economy of construction. Economical column spacings for flat slab designs for various live loads could have been worked out. In fact, much is to be said on the subject of economical design of concrete buildings. However, with the engineer governed largely by specific conditions, building laws, fiber stresses, etc., it would be difficult indeed to cover the subject, even in a general way, in a paper many times the size of this one. Therefore, it was thought best to give most of the space to methods of estimating the comparative costs of the principal members which go to make up a concrete building, and not deal specifically with the layout of the building. If the engineer has in hand the fundamentals of the cost estimating of his work he can solve the question of economy in his layout for himself.

Herein are contained the methods of calculating comparative costs of concrete buildings for the purpose of securing maximum economy. If the designing engineer is informed of the availability of the structural elements for reinforced concrete for the particular job he has in hand, and applies the principles as set forth in this discussion, there is no reason for the finished product being anything but a substantial structure in which material and expense were kept at a minimum.

Unit costs of labor and materials for all classes of building construction are constantly changing, and it is hardly to be expected that one whose business is not entirely estimating be kept well informed of the many fluctuations. However, it has been shown in this article that the designer does not have to use absolutely accurate unit costs in order to determine by comparative estimates, the relative economic standing of his designs. A review of the market conditions from time to time in a general way will give him enough information to revise his unit costs in order that his comparisons may show more accurately the true status of his work. The prices tabulated and used throughout this paper, as before mentioned, are much higher than the prices of two or three years ago. It is quite possible that two years hence they may undergo another change equally great, and it is left to the user of these prices to be governed accordingly. Again, mention is made of the fact that the designs and quantities used here to illustrate the methods of comparative cost estimating are in no way to be viewed from an engineering standpoint. These examples are intended to be of value only in so far as they serve for illustrative purposes and must not be considered as solutions to engineering problems. It is not within the scope of this paper to lay down rules for concrete design but rather define and illustrate principles by which the accepted rules may be economically applied.

In writing this paper it is not intended to try to make building estimators of all designers of concrete buildings. It is, however, hoped that it will arouse the interest of the designer to a keener desire to know the cost of his designs, and to promote the practice of comparative cost estimating among designing engineers. Such practice, if conscientiously followed, will be extremely helpful in enabling the designer to choose wisely, and at the same time imbue him with a new confidence that his work really contains the most economical methods to achieve his purposes. He will get a new perspective of his work and develop to a greater degree the capacity for thinking in dollars and cents as well as in stresses and strains. His work will become more interesting and his services of more value. It was with this in view that this paper was conceived, and the effort will be well worth while if this purpose has, even in a small way, been accomplished.

DISCUSSION.

Mr. L. J. Mensch.—The statement that "Unfortunately no rules, tables or data are at hand which will solve the problem of maximum economy in the choice of the various methods of concrete construction which may be used" cannot be allowed to pass without the writer's emphatic protest. Mr. Mayer, as a concrete expert, ought to know that there was published a book nearly ten years ago ("The Reinforced-Concrete Pocketbook" by L. J. Mensch) which deals with this very subject. There can be found the design of every column beam and slab in practical use, tabulated according to the permissible load or bending moment, and in the same tables are given the quantities of concrete, steel and form lumber required per lin. ft. or sq. ft. of each column, beam or slab. Practically all floor constructions in practical use, and only the most economical designs, are tabulated for column spacings from 12 by 12 ft. to 24 x 30 ft. and for total loads from 100 to 500 lb. per sq. ft., accompanied by the quantities of concrete, steel and form lumber per sq. ft. of floor area, so that the relative cost of various spans and for various floor loads can be found at a glance. Data for estimating the labor cost and cost of material and plant are also given, and by examples it is shown how to design and estimate the cost of buildings in the simplest manner and in a very short time. **Mr. Mensch.**

CONCRETE ROADWAYS FOR THE INDUSTRIAL PLANT.

By G. S. Eaton.*

A large industrial plant is in itself a thriving town with an exceedingly large population and traffic for the area covered. In this community the populace keeps passing back and forth day and night, with rush hours during labor shifts. Traffic continues between buildings, and to and from neighboring towns or cities. Workmen on foot, motor trucks loaded with merchandise, passenger cars, laborers with wheelbarrows—all must pass over the streets of this community, all will be hindered by dust or mud. For protection from fire, it must be possible to move apparatus with speed and certainty in the industrial plant, just as in any city or town.

Better sanitation is another important reason for the construction of hard-surfaced driveways—easily and cheaply cleaned—to replace the refuse-collecting earth or cinder roads. One improvement leads to another; heaps of rubbish and waste disappear when paving is done.

Even though the city pave to the gates of the plant, bad spots between the loading platform and the pavement may cut a truck's carrying capacity in two. Incidentally, repair bills mount up to an astonishing total when rough and muddy drives must be used. Deliveries on schedule are certain only when paved roadways make up the entire route, and since motor-truck delivery has already been established for local freight hauls wherever road conditions will permit, considerable expenditures are justified in preparing for efficient service. Such an impetus has been given this new means of freight handling that there will be no thought of lessening its use when war conditions give way to those of normal times.

But the material saving in hauling costs, improved sanitation, increased fire protection, and independence of rail transportation are far from being the only considerations leading to a decision for improvement with concrete roadways. The creation of a sentiment of loyalty and cooperation among the workers is largely dependent upon the presence of pleasing surroundings and the provision of simple physical comforts. High standards of personal efficiency cannot be expected where the first and last thing the employee sees at the plant is the inefficiency of poor roads.

The satisfactory pavement must offer low tractive resistance at all times; be easily cleaned or self-cleaning with the aid of rain; furnish an even yet non-slippery surface, and be attractive in appearance. To be economical, it must be low in first cost, durable, and require but little maintenance. How well concrete meets these requirements is too well known to need further comment. The best proof of concrete's value for paving is the surprisingly rapid increase in yardage constructed within the last seven years in the United States. How great this increase has been, and the importance to the cement world of this recent development in the use of concrete, is seen in the following table:

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* Assistant Division Engineer, Promotion Bureau, Universal Portland Cement Co., Chicago.

YARDAGE OF CONCRETE PAVEMENT PLACED IN THE UNITED STATES AND CEMENT USED IN SUCH CONSTRUCTION.

(For Seven Years Ending 1917.)

Year.	Concrete Pavement Constructed, sq. yd.	*Cement used for Concrete Pavement, bbl.	Apparent Domestic Consumption, bbl.	Percentage used in Concrete Pavements.
1911	1,439,000	480,000	72,577,000	0.7
1912	5,381,000	1,794,000	80,866,000	1.2
1913	7,594,000	2,531,000	85,810,000	2.9
1914	15,739,000	5,246,000	84,419,000	6.3
1915	18,598,000	6,199,000	84,369,000	7.3
1916	24,219,000	8,073,000	91,990,000	8.8
1917	21,771,000	7,257,000	†88,360,000	8.2

RAILROAD USES CONCRETE FOR SHOP ROADS.

Already many industrial establishments, large and small, have followed the example of city and rural authorities in adopting concrete for their pavement work. One of the larger road systems was that built by the Atchison, Topeka and Santa Fé R. R. last year. The officials of that company felt the need of improved roads in their shops at Topeka, Kansas. Here are located the principal shops of the system in a plant covering 120 acres. After exhaustive tests, especially on tractive resistance, the decision was made for improvement with concrete. In this plant, much use is made of small electric motors for hauling supplies and machinery on very low trucks. Wheels of small diameter such as are used in this work dig into any but hard surfaced roads (Fig. 2.) The new pavements have now been in service for several months with the result that a great saving has been made in expenditures for hauling. There are 14,200 sq. yd. in the system, and 7600 sq. yd. more have been authorized.

The extensive concrete road systems constructed by the Universal Portland Cement Co. in its plants at Buffington, Ind., and Duluth, Minn., will be described at some length later. Altogether, this company has had constructed approximately 26,500 sq. yd. of concrete pavement.

Following are some other companies which have concrete roadways in and around their plants.

Lincoln Paper Mills, Ltd., Merritton, Ont.
Moline Automobile Co., Moline, Ill.
Allis-Chalmers Manufacturing Co., West Allis, Wis.
E. I. du Pont De Nemours & Co., plant at Hopewell, Va., and Harrison's Works, Philadelphia.

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* Estimated on basis of 1 barrel to 3 sq. yd. of pavement.

† Estimated from figures for shipments.

United Fuel & Supply Co., Detroit, Mich.
Atlantic, Gulf & Pacific Co., Brooklyn, N. Y.
S. S. White Dental Supply Co., Princess Bay, N. Y.
Studebaker Corporation of America, Detroit, Mich.
General Chemical Co., Edgewater, N. J.
American Can Co., Hackensack, N. J.

In deciding on the layout of the road system, these factors, among others, should be considered:

1. Plan for minimum of paved area to serve as directly as possible all points of exchange.
2. Placing pavements so that they may be used as sidewalks.

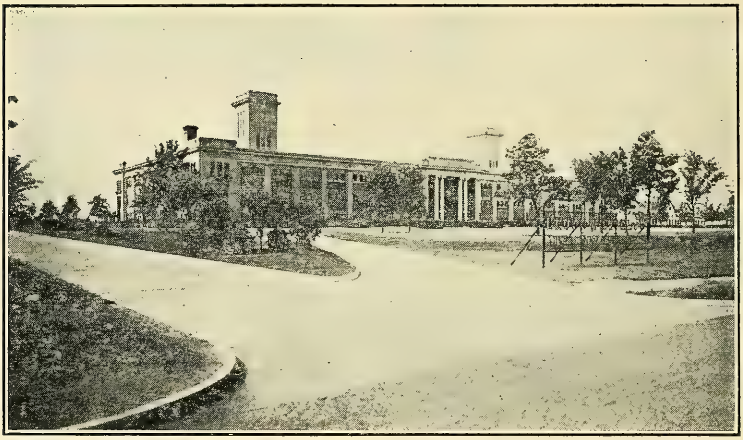

FIG. 1.—CONCRETE PAVEMENTS ADD TO ATTRACTIVENESS OF GROUNDS AT BROWN & BIGELOW'S PLANT, MINNEAPOLIS.

3. Adapting roads to existing yard levels to avoid unnecessary work.
4. Appearance of grounds.
5. Provision for expansion of road system with enlargement of plant.
6. Making pavements suitable for traffic by providing long radius turns, easy grades and ample paved width.

UNIVERSAL PORTLAND CEMENT CO. ROADS.

Time spent in making a complete study of conditions will be well repaid The experience of others is often of considerable value, and, therefore, the way in which some details of design were worked out by the Universal Portland Cement Co. in the paving work done in its Duluth and Buffington plants, during the last two construction seasons, will be given.

But first a few words as to the extent of this work should be of interest. Normally, a cement plant does comparatively little shipping except by rail.

Raw materials come in by rail and cement is shipped out in the same way. Yet, as a plain, economic proposition, the company found it worth while to pave the driveways in the plants with concrete. Complete systems of concrete roads have now been constructed at the Duluth, Minn., and Buffington, Ind., plants. The private system of roads at Duluth connects with the concrete pavement on Commonwealth Ave. which leads into the city. At Buffington, besides the 13,500 sq. yd. of pavement placed within the plant, there has been built an 18-ft. concrete road 3,600 ft. long connecting with streets of Indiana Harbor, from which point there are good roads spreading out over the Calumet District, an important manufacturing section. At both plants, these improvements make possible deliveries to extensive users of cement by truck direct, and in addition give an outlet to more railroads, should it be desirable to ship by other routes. The total yardage placed was approximately 26,500 sq. yd. Of this, the pavement in the plants—19,500 sq. yd.—was placed entirely by the company's forces. This is equal to the amount of pavement needed for 3.7 miles of 9-ft. road.

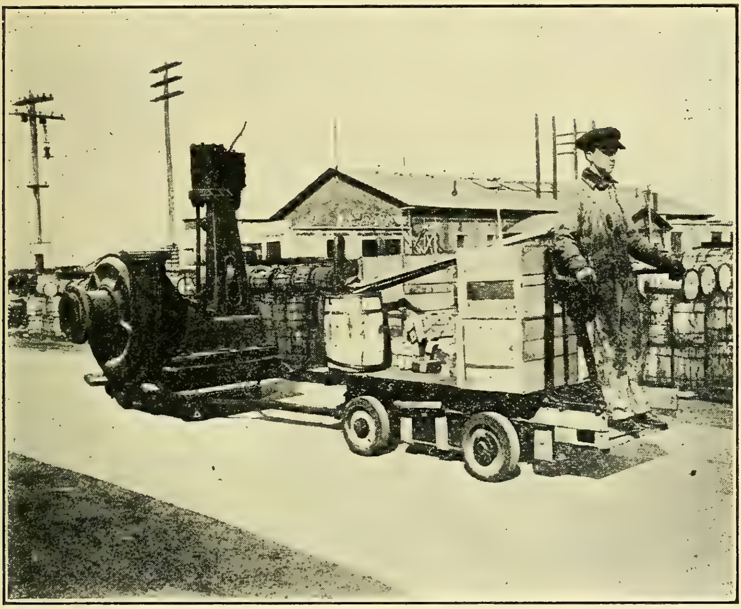

FIG. 2.—HAULING BY MOTOR ON CONCRETE PAVEMENTS AT SANTA FÉ RAILROAD SHOPS.

In designing a system of concrete roads it should be kept in mind that for economy pavements should not be wider than necessary for the traffic they will be called upon to carry, making reasonable allowance for the future.

With short distances between turn-outs, one vehicle can wait for a clear roadway on the drives that have light traffic only. Where pedestrian as well as vehicle traffic is expected, a 10-ft. pavement should be considered the minimum width for a single track road; an 11 or 12-ft. width is much to be preferred. Trucks are in operation today which have an overall width of 7½ ft.; such a machine might very easily get off the narrow strip of concrete, and, in bad weather, it would probably be stuck. Consideration should be given to the workmen who are to use the pavement as a sidewalk at the same time trucks are operating over it. Everything points to at least a 10-ft. pavement, and with much pedestrian traffic, an 11 or 12-ft. width. For a double-track roadway, at least 16 ft. is needed, and in view of the tendency to increase truck widths, preferably 18 ft.; 9 and 16 ft. were the widths used in the paving work inside the Universal Company's plants, but if the work were to be done over, these widths would be increased. The road to Indiana Harbor was made 18 ft. wide.

WIDTHS ON CURVES.

It must be remembered that the road widths just given are for straight sections only. On curves, unless of large radius, the pavement must be widened. At Buffington trucks weighing when loaded as much as 14 tons are being used to haul cement. These trucks are 7 ft. in width, measuring from out to out of rear wheels, and have wheelbases of 14 ft. (Fig. 3.) Such a vehicle cannot make a turn with an inside radius of 20 ft. 6 in. (minimum used at Buffington) on a 9-ft. pavement; the width on the curve should be increased to at least 12 ft., and, for safety, preferably to 14 ft. To allow two trucks to pass comfortably on a turn with an inside radius of 20 ft. 6 in. would require a width of some 27 ft. Larger trucks than these are being manufactured, and where their use is a possibility, curves should be widened more than the amounts given above.

In any case, where both axles are rigidly attached to the chassis, the width of single-track road theoretically needed may be found from the following formula (all dimensions in feet):

$$W = R_2 - R_1, \text{ where } R_2 = \sqrt{(R_1 + B)^2 + L^2},$$

W = width pavement,
R_1 = inside radius of curve,
R_2 = radius of outer front wheel,
B = out to out of wheels,
L = wheelbase of truck.

W should be increased sufficiently to permit practical use of the pavement. On double-track pavements, allowance must be made for the overhang both front and rear.

Stub ends must be within backing distance of turnouts or else so designed that trucks can turn around on them. At Buffington there were provided many wide areas at platforms and doorways. More than 50 per cent of the paved area in this system is in irregular areas, including curves and intersections.

The radii at intersections must be ample but not excessive, for a small increase of radius adds a large area of concrete. An inside curve radius of 20 ft. 6 in. was adopted as a minimum after considerable study of the operation of auto trucks; it is believed that any truck made today can turn with this radius if sufficient width of pavement is available. Whenever possible, however, larger radii were used on curves, but not at intersections in all cases. If a 25 ft. radius is used on intersections instead of a radius of 20 ft. 6 in., the area of the curved parts is increased by 20 sq. yd., or 50 per cent. Care must be taken, however, to provide sufficient width so that large motor trucks can make the turn, just as on curves; narrow pavements should be widened somewhat at both threeway and fourway intersections, where this minimum radius is used.

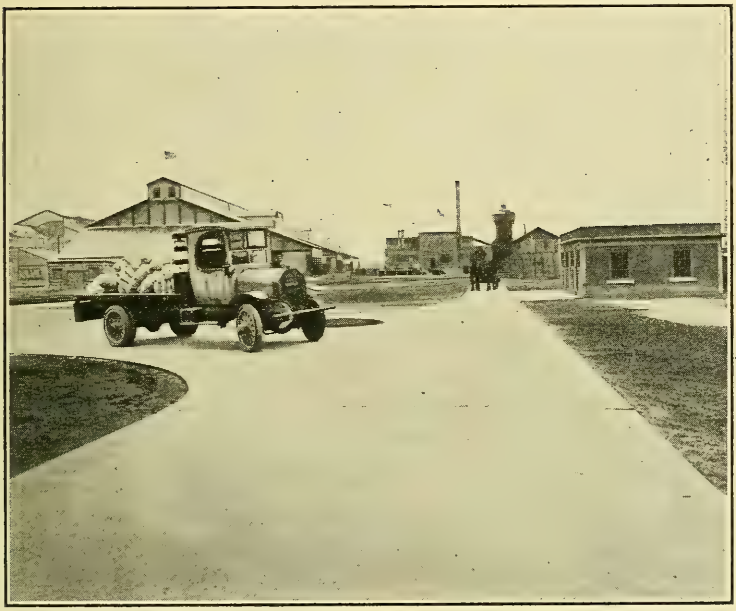

FIG. 3.—DELIVERING CEMENT FROM BUFFINGTON PLANT OVER CONCRETE ROADS.

Superelevation of Turns.

Curves were superelevated $\frac{2}{3}$ in. per foot of width of pavement; that is, 6 in. on a 9-ft. road and $10\frac{2}{3}$ in. on a 16-ft. road. This was done by raising the outer edge of the pavement the entire amount in the last 20 ft. before the point of curvature was reached, giving the full superelevation all the way around the curve. The inner edge of the pavement was carried around at grade.

For a superelevation of ⅔ in. per foot of width and with a radius of 25 ft. (the center radius of the sharpest turns on a 9-ft. roadway) a speed of 4½ miles an hour is entirely compensated for. At greater speeds the turn can be made much easier than would be the case without superelevation. Danger of getting too great a slope for horses and pedestrians limits the rate of superelevation that can safely be used.

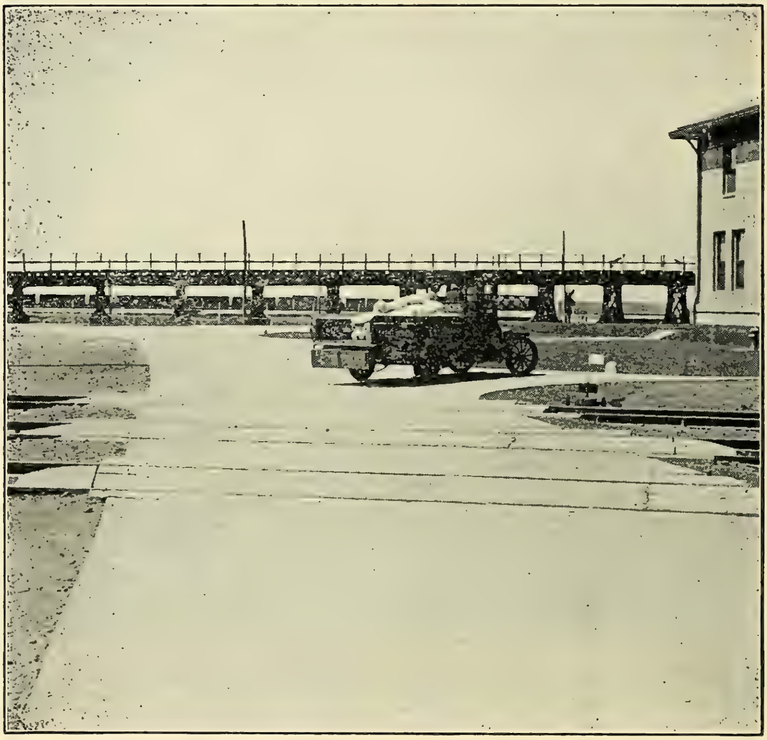

FIG. 4.—PRECAST CONCRETE SLAB RAILROAD CROSSINGS IN INDUSTRIAL PLANT.

Intersections, both threeway and fourway, were also superelevated to favor the vehicles turning, at a slight expense to those passing straight through. In constructing a threeway intersection, the inner edges of the pavement were depressed and the outer edge was raised. Where the straight side intersected the center line of the road at right angles to it, it was made 9 in. higher than were the mid-points of the inner edges. The straight side sloped down to the normal elevation 15 ft. past the point of

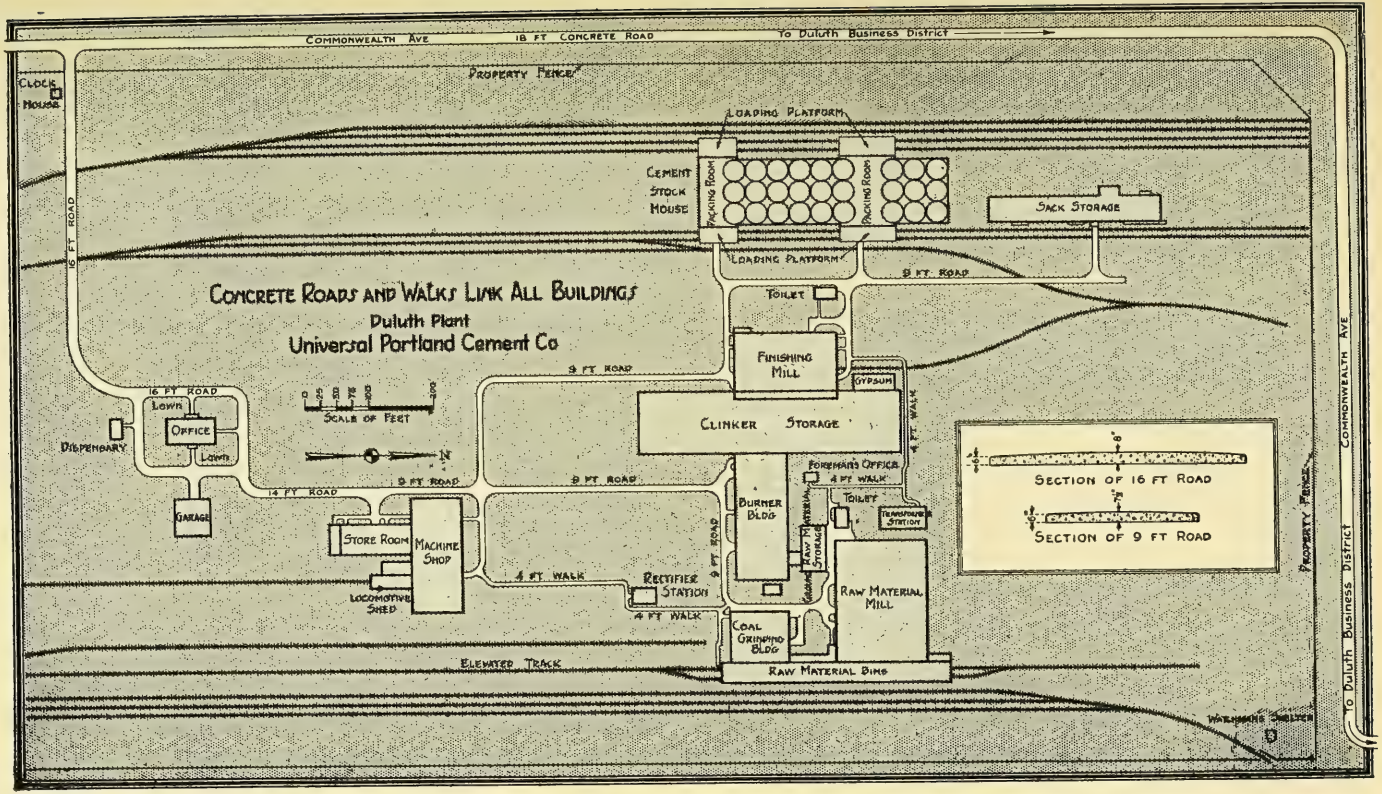

FIG. 5.—CONCRETE ROADS AND WALKS IN DULUTH PLANT, UNIVERSAL PORTLAND CEMENT CO.

curvature. From the straight side, the center line sloped to the center elevation of the right angle road at the point of curvature.

The center of a fourway intersection was made 3 in. higher than the center lines at the points of curvature and the centers of the inner edges were depressed 1½ in. Including the crown, this gave a total superelevation of 6 in. on the 9-ft. roads. In practice, it was necessary to modify these plans for securing superelevation but they may be taken as typical.

Especially for narrow roads and sharp turns superelevation is desirable, for it helps to make turning easier and keeps the motor car under better control. At fourway intersections on double-track pavements, however, superelevation is usually not to be recommended.

In the work of construction, standard practice was followed. Since the subgrade was of fine sand, no rolling was attempted, but it was thoroughly sprinkled before placing concrete. A 1:2:3 mix was used, with coarse sand and crushed limestone as the aggregates. Areas over 16 ft. in width were reinforced with ⅜-in. rods spaced 12 in. center to center both longitudinally and transversely. Premolded expansion joints without protection plates were placed at 25-ft. intervals. Finishing was done by the roller and belt method.

At Buffington, the 9-ft. roads were made 6½ in. thick at the sides and 8 in. at the center. On the 16-ft. pavements the thicknesses were 7 in. and 9 in.

PRECAST SLABS FOR RAILWAY CROSSINGS.

Another interesting feature is found in the use of precast concrete slabs for railway crossings. Usually, industrial tracks are poorly ballasted and maintained; unless the crossings are well constructed they will be higher or lower than the pavement adjoining. To avoid these difficulties, 22 crossings in the Buffington plant were reconstructed.

First the old tracks were torn up, a trench excavated, and then some concrete sidewalk slabs—no longer needed with the new pavements—placed upside down in the trench, a clear 6 in. below the bottom of the tie. Sawed ties were laid in slag ballast and a standard 85-lb. rail spiked to the ties; 45-lb. rails turned on side were used for providing flangeways.

At 20 of these crossings precast concrete slabs were placed between rails and between track and pavement. Most of the slabs were cast last winter in the burner building at Buffington. Concrete for the majority of them was mixed in a small paving mixer and they were cured indoors.

The slabs resting directly on the ties make an even, safe and durable pavement, yet one that can be easily removed for track maintenance. Although the first cost of these crossings is somewhat more than that for plank construction, they should outlast several of the latter type, proving more economical in the end.

In Cedar Rapids, Iowa, two crossings of a design similar to that used at Buffington have given excellent service for three years. These were constructed by T. F. McCauley, City Engineer, who has placed a third slab crossing this season. One of the crossings constructed in 1915 is half of concrete slabs and half of planks. Mr. McCauley states that it has been necessary to renew the plank portion three times, while the concrete has had no maintenance.

At two crossings in the Buffington plant where the track extends through a wide pavement, the construction for pavement between street car tracks was followed. The flangeway was secured by the use of a specially shaped wooden groover, and Kahn curb bars were used to protect the edge of the flangeway. On switch crossings at the Topeka shops of the Santa Fé Railroad, where concrete was placed between the rails, an angle was fastened next the rail to give the needed clearance.

While this discussion has been devoted chiefly to concrete roadways for large plants, pavements are of equal importance for small factories, lumber companies, and material yards. Many such concerns have already constructed concrete driveways; others are planning such work for this season. Industrial plants, large and small, need permanent pavements, both for the actual saving in transportation costs and for the effect on the efficiency of employees.

AN INVESTIGATION OF THE VERTICAL MOVEMENTS OF CONCRETE PAVEMENTS.

By John W. Lowell.*

That the American public is expecting more and more of its roads and pavements is indicated by greater use of highways, demand for better types of construction, and the mileage being built that could not have been conceived a few years past. Pavements considered highly satisfactory a few years ago are generally condemned today. Nearly every one has become so exacting of quality that a mere visual blemish is the object of criticism.

The concrete pavement in the short time it has been in use has been perfected to a remarkable degree, there remaining but one real serious criticism —that of longitudinal cracks.

Cracks occur not only in concrete pavements, but in all permanent hard-surfaced pavements, and so in discussing cracks, their causes and elimination, other types of hard-surfaced pavements are included.

Cracks are objectionable; just to what extent is problematical and depends largely on the viewpoint of the critic. To the layman they have a psychological effect; to the engineer they indicate non-continuity of surface, permit concentrated wear and require maintenance; to the promoter of a competitive type of pavement, they offer opportunity for criticism.

What it is worth to the public to eliminate cracks depends largely on how badly it wants them eliminated. To the engineer, it is a problem in economy. Suppose, for the sake of argument, that any engineer was asked to build a concrete or other hard-surfaced pavement in which longitudinal cracks would not occur, just how would the problem be solved, and how many different solutions would result? There would, no doubt, be several, according to the number of discussions that have appeared during the last two years in engineering periodicals. These solutions would largely be based on assumption and not on scientific fact.

Before cracks can be eliminated it is pertinent to know of their existence and cause. In climates where freezing occurs, we have found, from general observation of pavement slabs having no greater dimensions than about 60 ft. the following to be true:

1. Transverse cracks do not occur as frequently as longitudinal cracks.
2. For widths over 18 ft., frequency of longitudinal cracks is apparently unaffected by width of pavement.
3. Cracks seldom occur in pavements lying on sand or other porous subgrades having natural drainage.
4. On clay or heavy loam soils, longitudinal cracks are of frequent occurrence.

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5. The more compact the subgrade soil, the more frequently cracks occur.

6. Cracking of pavements on clay or heavy loam soils cannot be entirely prevented by artificial drainage.

An exceedingly conclusive investigation conducted by the Office of Public Roads and Rural Engineering, reported in its Bulletin No. 532, is really the first scientific study made on this subject. Two facts were practically established by extensive investigation of the horizontal movement of concrete pavements. They are:

1. Transverse cracks are usually caused by contraction.
2. The effect of friction between pavement and subgrade is practically negligible.

EXPERIMENTAL STUDY MADE.

Early in the fall of 1916, the Universal Portland Cement Co. undertook to learn something definite about the vertical movements of concrete pavements. After considering several locations at which to make this investigation, a stretch of newly-built concrete pavement 3325 ft. long and 40 ft. wide with integral curbs on Sheridan Road, Kenilworth, Illinois, was selected. This pavement, 7 in. thick at the sides and 10 in. thick at the center, was of two-course construction, the base being a $1:2\frac{1}{2}:4$ pebble concrete and the wearing course a 2-in. thickness of $1:1:1\frac{1}{2}$ concrete in which the coarse aggregate was granite from $\frac{1}{4}$ to $\frac{3}{4}$ in. Reinforcement consisted of No. 27 triangle mesh, wire fabric, weighing 41 lb. per 100 sq. ft., placed between base and wearing course. One-half-in. round steel rods 14 ft. long, placed transversely 12 in. apart 2 in. above the bottom, were used on the east half of the street as extra protection, a large sewer ditch having been built along this side for the full length of the improvement two years prior.

The pavement having been completed before deciding to make our investigation, we were unable to make borings to determine the character of the subgrade. The engineer on the construction did, however, keep observation notes of the general appearance of the subgrade, and these records present a general description of the character of at least the surface subgrade soil. These notes were checked as well as possible by scattered borings along the sides of the pavement. From Slab 1 to 38, the subgrade may be described as a spongy tough compact clay; from 39 to 55, sand loam with spongy under stratum and compact crust; from 56 to 70, clay and loam, with some fine sand, the ground being spongy with sand below; and from 79 to 87, the south end of the pavement, sand greater than 3 ft. in depth.

The natural lay of the soil, was, of course, broken up under the east half along the sewer, the excavation for which was about 2 ft. in from the east edge, from 8 to 10 ft. wide and 16 ft. deep. The subgrade surface appeared to be more compact on the west than on the east half, which was badly cut up and rutted by teams. The ruts, however, were smoothed out as well as possible with sand, gravel and earth before concrete was placed.

The pavement was well built, excellent aggregates being used in correct proportions and well mixed. Proper precautions were taken in the hardening of the concrete and the last slabs laid were 38 days old before the air temperature dropped to 32° F.

Just after completion of the pavement preparations were made for observation and study of the vertical movements of each of the 87 slabs, by a series of precise levels at three points on each side of every expansion joint. The points were taken 4 ft. from each curb and at the center, and permanently located by small holes drilled in the steel joint plates. The initial reading of each joint accurate to 0.1 mm. was taken on Oct. 27 and 28, 1916, and each reading reduced to 0 elevation, so that future readings would indicate directly the movement above or below the original position. Subsequent readings were not taken at definite intervals, but when weather conditions indicated that a change might be taking place in the subgrade. Eight readings were taken during the 12 months, the last on Oct. 25 and 26, 1917. Coincident with each reading a survey was made of the pavement surface and all cracks appearing were plotted.

Temperature and rainfall data was collected from the United States Weather Bureau Station at Chicago, 15 miles south of the pavement and in the same relative position with reference to Lake Michigan.

WHAT HAPPENED TO THE SLABS.

After collecting this mass of data, there being 4176 elevations taken, each requiring three rod readings, much work was necessary to put it into such shape that would permit of study, not only of individual slabs, but of groups, and the whole. After several systems of grouping were tried the one shown by Fig. 1 was adopted. From this form many combinations were made. All measurements and computations were made in millimeters, but for the text of this paper, these values have been converted into fractions of an inch.

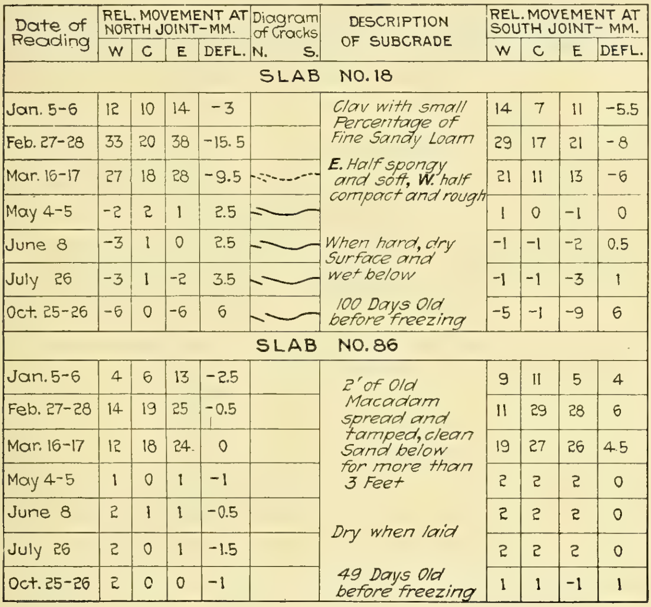
Of 86 slabs, 56 cracked. Thirty-eight which cracked deflected at one or both joints more than $\frac{7}{16}$ in. in the 32-ft. width between outside points, 18 less than $\frac{7}{16}$ in. and of these, eight were between $\frac{3}{8}$ and $\frac{7}{16}$ in. while four slabs having as great deflection failed to crack. Six of the ten remaining slabs which cracked, having maximum deflection of from $\frac{3}{8}$ to $\frac{3}{16}$ in. evidently cracked from abnormal settlement of a portion of the subgrade below the original level, leaving four slabs, one of which cracked at $\frac{3}{16}$, one at $\frac{5}{16}$ and two at $\frac{9}{32}$ in., which defy explanation, unless the deflection had been greater at some period between readings.

Every slab reaching a deflection of more than $\frac{7}{16}$ in. cracked and there were 38 out of 86, so it is safe to assume that such deflection in 32 ft. of width is sufficient to crack any pavement slab of this width built in accordance with present specification and practice. It is also evident that even greater deflection than $\frac{7}{16}$ in. can be expected when the subgrade is clay or loam undrained.

Had the maximum deflection been less than $\frac{11}{32}$ in., practically no longitudinal cracks would have occurred, and where the subgrade has a uniform moisture content the deflection would probably never be this great. This is substantiated by those slabs on sand subgrade.

Thirteen consecutive slabs at the extreme south end of the pavement on sand subgrade acted quite differently and more uniformly than the others. The average upward movement was $\frac{11}{16}$ in. on the west and $1\frac{1}{16}$ in. on the east side, while the center went $\frac{31}{32}$ in. The greatest rise of any point was $1\frac{15}{32}$ in. The movements were so uniform, however, that the greatest deflection was $\frac{1}{8}$ in. Unlike the other slabs there was practically no settlement below the initial reading, the greatest recorded being $\frac{3}{32}$ in.

The movements of two typical slabs are shown on Fig. 2, together with the rainfall and temperature charts. Upon freezing the sides and center of these slabs at first began moving up uniformly. During this cold weather considerable snow fell and then during the first week in January a thaw occurred followed by extremely low temperature for the next eight weeks. In this period the sides of Slab 18 on clay subgrade rose more rapidly than the center until a maximum deflection of $\frac{5}{8}$ in. occurred, while Slab 86 on sand subgrade rose but $\frac{6}{10}$ as high, there being practically no deflection.



Date of Reading	REL. MOVEMENT AT NORTH JOINT-MM.				Diagram of Cracks N. S.	DESCRIPTION OF SUBGRADE	REL. MOVEMENT AT SOUTH JOINT- MM.			
	W	C	E	DEFL.			W	C	E	DEFL.
SLAB NO. 18										
Jan. 5-6	12	10	14	-3		*Clay with small Percentage of Fine Sandy Loam*	14	7	11	-5.5
Feb. 27-28	33	20	38	-15.5			29	17	21	-8
Mar. 16-17	27	18	28	-9.5		*E. Half spongy and soft, W. half compact and rough*	21	11	13	-6
May 4-5	-2	2	1	2.5			1	0	-1	0
June 8	-3	1	0	2.5		*When hard, dry Surface and wet below*	-1	-1	-2	0.5
July 26	-3	1	-2	3.5			-1	-1	-3	1
Oct. 25-26	-6	0	-6	6		*100 Days Old before freezing*	-5	-1	-9	6
SLAB NO. 86										
Jan. 5-6	4	6	13	-2.5		*2' of Old Macadam spread and tamped, clean Sand below for more than 3 Feet*	9	11	5	4
Feb. 27-28	14	19	25	-0.5			11	29	28	6
Mar. 16-17	12	18	24	0			19	27	26	4.5
May 4-5	1	0	1	-1			2	2	2	0
June 8	2	1	1	-0.5		*Dry when laid*	2	2	2	0
July 26	2	0	1	-1.5			2	2	2	0
Oct. 25-26	2	0	0	-1		*49 Days Old before freezing*	1	1	-1	1

Zero Reading Oct. 27-28, 1916

FIG. 1.—VERTICAL MOVEMENT OF TYPICAL SLABS, ITS EFFECT, AND NATURE OF SUBGRADE COMPILED.

These concrete slabs were exactly alike, precipitation and temperature was the same, the only difference being in the character of the ground on which they lay. Since the subgrade is of inert material of similar nature, the only difference is in the size of the dirt grains. Without going into details of the relation of moisture in soils to their volume, it is permissible to state that the finer the dirt grains the greater the resistance to passage of water by gravity, the greater the change in volume for a given percentage of water contained and the greater the expansion of the mass on freezing. This statement is borne out by referring back to Slabs 18 and 86, the movements of which are shown on Fig .2, No. 18 being on fine-grained and No. 86 on coarse-grained soil.

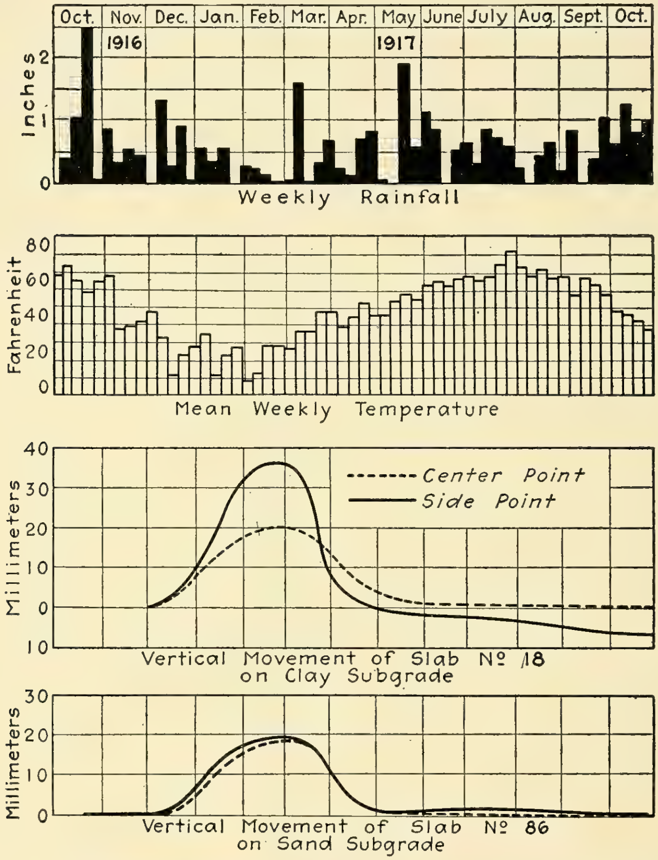


FIG. 2.—VERTICAL MOVEMENT OF SLABS COMPARED WITH REFERENCE TO RAINFALL AND TEMPERATURE.

The assumption that the subgrade freezes deeper at the sides than under the pavement center is wrong, for concrete has about the same coefficient of conductivity as moist earth. In another investigation we have confirmed this by measuring the depth of frost at the sides and under pavements.

Deductions as to Behavior.

Thus far we have facts. Now, let us see if from these facts we can explain the cause of this action that we have so accurately measured.

The moisture content below the central portion of a pavement will probably never be much less than when the pavement is laid because there is no surface from which it can be evaporated. Since surface water from each side is conducted under the pavement both from static head and capillary powers of the soil, the moisture content will continually change from an almost uniform degree throughout to a maximum at the sides. Should the soil be compact a uniform condition is not probable at any time, while in sand the water has less resistance to its passage and so reaches a uniform condition quite rapidly. Fig. 3 is a study of the probable distribution of moisture under pavements on clay or loam subgrades.

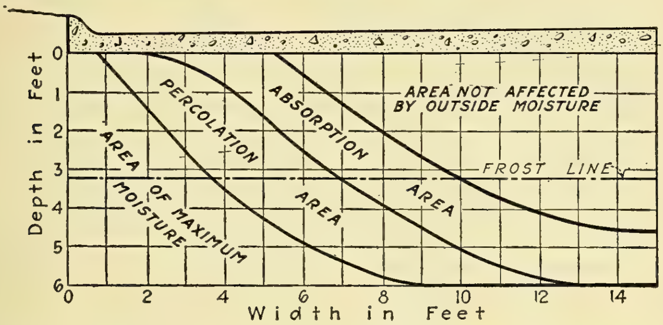


FIG. 3.—DISTRIBUTION OF MOISTURE PREVAILING IN UNPROTECTED CLAY AND LOAM SUBGRADES.

Upon freezing the soil expands in proportion to the moisture content and since it cannot expand laterally it must expand vertically. The greater the difference of moisture from sides to center, the greater the distortion of the pavement slab. Snow falls upon the ground and afterwards a thaw occurs, melting it. If the soil is not fully saturated the water sinks down and spreads out under the pavement as far as it can go, by virtue of head and capillary powers, until it freezes in the voids at some depth where the temperature is still below freezing. In this way an impervious stratum is formed through which the water will not pass. The remaining surface water is afterwards held above this stratum, and this soil rapidly becomes more water soaked. If there is enough surface water the soil finally becomes saturated as far out as the water can spread by gravity and then the water content extends farther under the pavement, decreasing as the capillary power diminishes.

This action is finally ended by return of cold weather and subsequent freezing of all moisture accompanied by expansion, which being restrained laterally acts vertically, but the vertical movement is proportional to the water (frozen) content above that amount required to fill the voids.

The vertical movement or heaving naturally is greatest at the sides where moisture is a maximum and decreases toward the center.

Concrete slabs are carried with the subgrade, wide slabs conforming to its shape because they do not have sufficient stiffness to carry their own weight. In this way distortion occurs, and when the deflection is such that the concrete on the extreme fiber exceeds its tensile strength a crack develops following the line of greatest deflection.

Pavements, regardless of width, should heave the same distance at the sides. Narrow pavements, those less than 16 ft. in width, should heave nearly uniformly because the moisture extends more uniformly toward the center. Pavements about 25 ft. in width should develop the greatest vertical curve, and above this width the curve should decrease with increasing width. It has been demonstrated, however, that pavements 40 ft. wide crack, which indicates that there is not enough decrease to bring the extreme fiber stress within safe limits, so this phase need not be considered.

If cracks are to be eliminated, there are but two general solutions to the problem. One is to overcome the conditions as they exist by slab design; the other to eliminate the conditions by controlling the moisture content. The first scheme could no doubt be carried out in several ways all of which would undoubtedly be expensive. An interesting and possibly less expensive solution is suggested by these experiments.

Variation in moisture can be eliminated by keeping water from percolating under the pavement. Although this is not a new theory it has as yet not been proved a success. Much money has been spent in vain on drain tile with and without porous backfill along the sides of the pavements, and why? Simply because they do not prevent the water from getting under the pavement either by pressure or capillary action. If results are to be expected, there must be an impervious underground wall along each side of the pavement extending down below frost level, and where ground water is present at least one foot lower to keep capillary water below frost.

The nearest we have yet come to the suggested type of construction to eliminate heaving is the old style 36-in. deep curbing with drain tile laid outside, and, no doubt, such construction is satisfactory where ground water is well below the street surface.

A construction that might be as effective and cost less would be a $\frac{3}{8}$-in. thickness of continuous jointed or lapped sheets of bituminous material such as is used for pavement joints, forming underground walls 4 or 5 ft. deep along each side of the pavement with 6-in. drain tile laid on a good grade just outside the base. Results that can reasonably be expected in clay or loam are illustrated on Fig. 4, in comparison to the study of probable conditions where no impervious wall exists.

Such construction would generally cost less than $1.25 a lineal foot, all of which need not necessarily be in addition to the present cost of concrete pavement for a considerable saving might be made on the pavement slab itself, in that reinforcing might be eliminated, thinner slab used and joints placed farther apart.

No doubt but that the elimination of longitudinal cracks would remove the last real objection of the layman to concrete pavements, and in some suburban locations this would be quite advantageous regardless of expense. To the engineer who has made a study of concrete pavements, longitudinal cracks do not seem serious and so far from experience there is no indication that they have any appreciable effect on the life or quality of the pavement.

However, cracks require maintenance and it is only reasonable to assume that maintenance will increase as pavements grow older. If the elimination of cracks would add a small additional life to pavements, this would represent a saving. If the reduced cost of pavement metal, saving in maintenance and increased life when capitalized should equal or exceed the cost of an impervious wall with tile drain, then such construction is worthy of consideration.

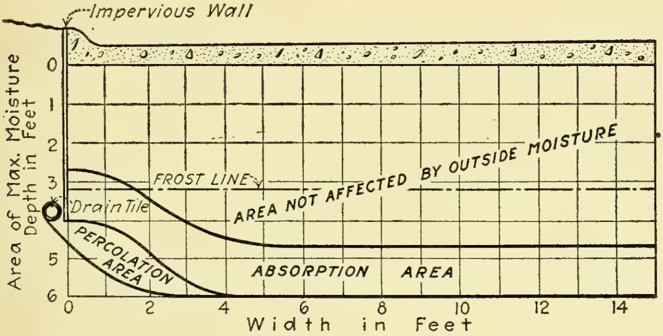


FIG. 4.—EFFECT OF IMPERVIOUS WALL UPON DISTRIBUTION OF MOISTURE IN SUBGRADES.

In concluding, I wish to extend thanks to Prof. William H. Burger of the Civil Engineering College at Northwestern University, Evanston, Ill., for his hearty cooperation in conducting this investigation, for it was he who made all the level measurements. This investigation is still being conducted on the Kenilworth pavement, and also an independent investigation was begun last fall on Sheridan Road, Winnetka, Ill. On this pavement, we started in time to obtain subgrade borings before the pavement was laid and are conducting an analysis to determine the effect of moisture and freezing on various soils.

HOW TO GET THE BEST SURFACE ON A CONCRETE ROAD.

By A. H. Hunter.*

In spite of the excellent riding qualities of pneumatic tires, together with all the experience and skill incorporated in the design of body, springs and wheel base of our automobiles, the automobile public is demanding improvement in the surface conditions of our concrete pavements. I do not feel that this is because previously constructed pavements were so decidedly inferior, but rather that with the coming of long stretches of improved surface, the novelty of riding on concrete diminishes. The traveler traversing longer units than before becomes more weary and with the accompanying fatigue comes criticism of surface imperfections hitherto unnoticed.

Any road surface, however durable, can scarcely hope to be ultimately successful for use in our through highways unless this wearing surface presents a minimum of jar and vibration. In a properly mixed concrete, such as is used in concrete road work, we have a plastic material susceptible of being molded into a nearly perfect surface. How closely we are to approach this theoretical plane surface depends upon the care and effort put forth by the workman in finishing. Upon workmanship depends largely the riding quality of surface secured.

The impression is not to be gained, however, that proper design is not essential. It would be utter folly to neglect drainage, crown, or both horizontal and vertical alignment, but fortunately, practice is becoming so standardized that most highway engineers recognize these important features in design. It is most infrequent to find a concrete pavement the poor riding qualities of which may be attributed to poor engineering.

If we are to avert a repetition of past errors, certain construction features must be carefully watched. These features to be considered from their significance as outlined in the specifications as well as their application by the layman.

Forms must be Rigid.

Too often the layman looks upon the side forms as merely a necessary convenience for holding the concrete, thus losing sight of the fact that the proper grade and alignment of these forms are highly essential if a smooth surface is to result. Not infrequently I have visited construction work and found adjacent form boards out of both grade and line. It is probable that in so far as line is concerned, the earth shoulder will later absorb a fair amount of irregularities, but it generally follows that a form setter who does not take enough pride in his work to properly align the boards is equally careless in setting the edge to the proper grade.

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* District Engineer, Division of Highways, Department of Public Works and Buildings, State of Illinois.

In early concrete pavements, wood forms were used exclusively. These, when properly used and well taken care of, have given excellent results. When in the hands of careless workmen, they soon warp, split and when not properly cleaned, cause frequent damage to the edge of the green pavement. When placed to accurate grade and alignment they must be thoroughly staked. Too few stakes is fatal. If these forms are not rigid, waves, bumps and depressions inevitably occur in the completed surface.

Once I heard criticism of the large vibratory finishing machines from the standpoint that they impose a larger load upon side forms than was thought advisable. Personally I do not take it as a criticism of the machine but rather of the forms. Make them rigid and the template properly used will cut the surface to the desired shape.

Steel forms are coming into general use. It is probable that specifications in the near future will express a decided preference for them. Many contractors, in an effort to save, purchase too light a weight, with the result that they are soon bent, twisted and unfit for use. In spite of preference for steel forms, they are not to be recommended for short, vertical curves or transverse curves of any character. They are too rigid to adapt themselves to the smooth outline necessary in a properly staked curve.

If we are to appreciably improve the surface of our concrete roads, it becomes necessary for the engineer to see that forms are accurately set to grade and alignment and maintained there. They are the only framework possible in a road. They are as essential as the formwork of a building. They are the guide lines of the pavement.

TEMPLATES NEED CONSTANT WATCHING.

After the concrete is placed within the forms, it should be struck off with a template having a length slightly greater than the width of the pavement. An excess length of possibly 18 in. will suffice for average highway work on pavement 15 to 18 ft. in width. This template to be moved by a combined longitudinal and crosswise motion.

Do not make the common error of cutting the arc of the template to the same dimensions as indicated on the cross-section. It should be cut ¼ in. to ⅜ in. deeper than the theoretical crown; otherwise, the resulting surface will be flat. This correction should vary as the width of the pavement as well as general conditions surrounding the job. The amount of water generally employed would be a factor for it is apparent that soft, mushy concrete would flatten out more readily than that containing a lesser amount of water. This template should be shod with a steel edge, using countersunk screws.

Templates may be made from either one or two boards. If two boards are used, they are held apart some distance by small struts. This style insures a more uniform bearing on the forms but is usually heavy, hard to handle and has a tendency to leave bumps or depressions, as the workmen are reluctant to move it back and restrike the surface.

The writer has seen good results obtained by using a heavy, double-edged steel template drawn by a cable, operated by a drum connected with the engine of the mixer. There is a general tendency to supplant man power wherever possible.

In moving the template along the forms, a slight excess of material must be in advance of the cutting edge at all times. Failure to do this will result in depressions or holes. Do not permit the template to ride on stone or gravel on top of the side-forms. In working on grades even as low as 3 per cent, repeated striking must be done if a smooth surface is to result. The concrete being plastic and of more or less non-uniform water content, will flow down the grade. This flow is not uniform but is given to taking the form of alternate humps and depressions which can only be prevented by repeated and continued striking until initial set takes place. Keeping the amount of water in the mix as low as possible assists materially.

Satisfactory results are obtainable only after pains are taken. The men operating the striker have to have an interest in their work or surface defects will occur. Watch the template! Check it up every few days, making sure that it has not warped or gone out of shape.

This matter of crown of pavement was forcefully impressed upon me several years ago, in connection with the construction of my first concrete road. The template was composed of two striking edges, several inches apart, covered with a plain board on top. At night, after work, it was washed off and placed on the forms behind the mixer. This afforded the night watchman (who, unfortunately for us, was of liberal proportions), or his visitors, a convenient seat. Continued use for this purpose resulted in a reduction in the crown of pavement by approximately $\frac{3}{4}$ in. This is no large amount, but it is apparent on pavements designed with a minimum of curvature. Frequently too a template, properly designed and cut to the required arc, develops a sag due to its own weight when placed in position on the forms. In general, templates—home-made devices especially—are not to be trusted. Good surfaces may be obtained, but it is only by exercising care and judgment that satisfaction is secured.

EXPANSION JOINTS.

In the first concrete road construction expansion joints were used in varying designs. Many otherwise good pavements had their riding qualities much impaired by these poorly placed joints. Unfortunately, highway engineers have not yet agreed among themselves as to what device is best suited to serve the purpose. Until standard practice develops some uniformity, we must expect to have continued experiments with occasional pieces of pavements unsatisfactory.

Generally speaking, any joint or holding device which does not permit the free use of the template is to be condemned. Not that excellent surfaces cannot be secured, but rather that it necessitates special care for the finisher. The elimination of expansion joints altogether, or possible acceptance of blind joints, will do much to improve the riding qualitise of our concrete surface.

FINISHING SURFACES AS RECENTLY DEVELOPED.

Naturally enough, concrete pavements were first finished with the same tools as sidewalks or building floors. The public approval of a wood float surface led to the general use of a small wood float, 12 to 18 in. long, operated by hand. It was possible for the contractor, at small cost, to finish by hand in a most satisfactory manner. It was a tiresome job for the finisher, operating from a cross plank or bridge. Too frequently the work was intrusted to unskilled men who probably slighted much of the surface, at the same time probably working holes, depressions or flat places at the more accessible portions of the surface. Many excellent surfaces have been obtained in this manner, but substitution of unskilled labor by the contractor in an effort to effect a small saving, together with the drudgery involved, led to development of easier and better methods

Following the hand float came the easier but unquestionably poorer experiment of placing a small wood float at the end of a long handle. This being operated from sides of the road, permitted the omission of a bridge. It was applicable to narrow pavements, being used with some success on roads up to 12 ft. in width. In general, the surface secured was much inferior to the hand method. The central portions of the pavement were neglected with the result that too frequent evidence of laitance, dirt or surface defects occurred near the center.

Engineers had recognized the importance of keeping the quantity of water required in the mix to a minimum but under working conditions a small excess had been permitted for the reason that concrete of proper consistency was difficult to work. Incidentally, the mixer with spout delivery required an excess of water. Several years back, experimenters in the States of Oregon and Michigan perfected mechanical devices for striking, compacting and finishing concrete, but these, while possessing merit, were heavy, costly and unsuited to operation on short, isolated sections.

It is only recently—not farther back than the construction season of 1916—that two simple operations, developed in different parts of this country, have been brought forward that when combined, resulted in radical changes in specifications and methods of finishing. I refer to the combined use of roller and belt in finishing concrete surfaces.

To an engineer of Macon, Ga., we are indebted for promoting the use of a small hand roller, some 8 in. in diameter and about 6 ft. long. This was operated transversely across the pavement by a handle. The old adage that "necessity is the mother of invention" certainly holds true to the method developed for smoothing concrete surfaces. It is told that a contractor in Michigan, during the summer of 1916, was inconvenienced by a strike which occurred among his gang after considerable concrete had been placed, struck off but not finished. It seems that in desperation, he ripped strips of canvas from his tent, dragged them over the surface in a desperate attempt to place the concrete in a condition satisfactory to the engineer and prevent possible loss. The results were astonishing; so much so that hand finishing was afterwards dispensed with and a canvas belt substituted.

During the construction season of 1917, the engineering organization of the State of Illinois, Department of Public Works and Buildings, Division of Highways, used their influence with contractors and secured their cooperation in experiments sufficient to warrant the Department in incorporating this belt-roller finish in the specifications for concrete highways, edition of March, 1918. These specifications representing the consensus of opinion of the entire engineering organization, are based upon experience gained throughout the State of Illinois, with different aggregate, different workmen and varying general conditions encountered in actual practice.

HOW TO USE THE ROLLER AND BELT.

The entire operation of finishing specified to consist of three beltings and two rollings, beginning with a belting and alternating until the final belting, the belts to be of a canvas or rubber composition, about 2 ft. longer than the width of the pavement. For the first two operations, a width of at least 10 in. is recommended. This belt also should be fairly heavy but pliable enough to conform to the crown of the pavement. No irregularities should be permitted in the edges, for these will cause ridges or depressions in the plastic surface. In both the first and second belting, the operation should consist of long, transverse strokes combined with a relatively slow, longitudinal movement. After the second rolling, the concrete is to be given a final finish, just previous to the initial set of the concrete. The time interval between placing the concrete to final finish is variable, depending somewhat upon the amount of water in the mix but largely on weather conditions. In other words, a highly satisfactory surface can be secured by final belting when the film of surface water has just disappeared. In the last operation, a light canvas belt about 6 to 8 in. in width, used with short, transverse strokes but with a sweeping longitudinal motion, will produce a uniformly gritty surface.

A satisfactory roller may be made with diameter of 8 to 12 in. and a length of 4 to 6 ft., having a total weight of $\frac{3}{4}$ lb. to 1 lb. per linear inch of roller. For roads up to 15 ft. in width, it may be operated by a long, light handle. Pavement of greater width requires the use of ropes in place of the handle, operated by a man on either side of the pavement. The first belting following immediately after the strike board, is in turn followed by the roller. The roller being advanced to the opposite side from the operator, is returned in the same track then moved forward for half its length and operated as before. The roller is thus advanced along the length of the road, each unit of surface receiving four separate treatments. A trip over in the manner described constitutes a rolling.

The surface should again be belted and, after a lapse of 15 to 20 minutes, again rolled. This time limit must be somewhat variable and left to the option of the resident engineer. Very wet mixing or cool weather encountered may make it desirable to recommend an additional rolling before final belting. In explanation of the foregoing ideas incorporated in specifications, a word of caution in regard to belting is necessary. The first belting is done to smooth up the surface only, remove slight irregularities and facilitate the removal of the surface water in advance of the roller. In general, belting should be a minimum for continued working of the surface results in an excess of mortar. Present practice favors the wear coming upon the large aggregate with only enough mortar present to securely bond the stones and prevent voids.

Possibly the foregoing description of finishing may seem complicated but in reality the physical labor involved is relatively small—much less than required in the old wooden float method. The cost per unit of surface is materially reduced. Surface imperfections are a minimum. The roller eliminates many bumps, by pushing them into the depressions or placing them parallel to the line of traffic where they are unnoticed. Each unit of area receives the same amount of attention and in the end a uniformly gritty surface is obtained, far surpassing in texture, uniformity and riding qualities, the original hand methods.

Roads constructed by roller method have shown an apparent reduction in the transverse cracks. Later uncompleted experiments in laboratory tests reveal the fact that with the rolling and removal of water comes an increase in concrete strength.

In mentioning the recent developments of surface finishing, it should not be taken that the roller-belt method represents conditions that cannot be improved upon. For simplicity and low cost of operation, it is at present unequalled and its use is to be recommended in the construction of small, widely separated sections of work. It appears probable that future developments in concrete pavements will have largely in mind the proportioning of aggregate so as to produce the greatest possible strength. In connection there will come equipment which by rolling, tamping, or otherwise will increase the density and strength at the same time securing all the advantages of a roller-belt finish.

SOME EXPERIENCES ON THE VERMILION COUNTY CONCRETE ROADS.

By P. C. McArdle.*

When Vermilion County, Illinois, determined to build its system of county roads by means of an issue of bonds in November, 1914, it had already been determined after consultation with the State Highway Department and the State Highway Engineer of Illinois, that the types of road on this system should be either of portland cement concrete or of cement grouted brick on concrete base.

The superintending of the actual design and construction of this system of roads devolved upon me. I shall not take your time up with any academic discussion of the ordinary methods of concrete road building, but will discuss only some of those of design, construction and inspection of materials and workmanship, which are distinctively those developed in this county.

In 1916, our designs of right angle turns provided for a minimum of 50 ft. radius with 4-in. superelevation on the center of curve tapering to zero at P. C. and P. T. respectively. Experience has proved that the minimum should be 100 ft. radius and with a minimum of 6 in. superelevation carried from P. C. to P. T. and tapering on tangent to zero. On T intersections curves are carried where possible, 100 ft. radius and tapering as before, leaving an angle unpaved between the roads, as traffic on roads previously built showed no sign of using this angle. (See Fig. 1.)

The original design required that an edger of 2-in. radius should be used on each side of the concrete pavement and a $\frac{1}{4}$-in. radius upon edges adjacent to felt joints. Experience has shown that for the protection of the concrete pavement both at the joints and on the sides, these have proved wise provisions.

Our entire system of roads was originally divided into nine divisions of approximately 19 miles each, and contracts were let accordingly. A division engineer was assigned to take charge of each division and with such assistance as he might need to properly carry on the work of engineering and inspection, and he was required to give his entire time and attention to the work. Each division engineer was furnished with a complete set of sieves, scales, necessary to make field inspection of coarse and fine aggregates, as well as transit fitted with level bubble, etc., and if in doubt, could refer any sample to our laboratory at Danville.

On several of these divisions, crushed stone was used. In these cases, each carload of aggregate was loaded in the presence of our inspector and a test made thereon, before material was shipped, and as a result there is on file in our office at Danville a test report of 1860 carloads of such material delivered during the years 1916 and 1917 which show astonishingly uniform results. The aim of the department was to completely inspect all material called for in the specifications and contract thoroughly. Reports of all inspections were required to be made by the division engineer to the office of the superintending engineer and all such reports are there kept on file.

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* Superintending Engineer, Vermilion County Roads, Danville, Ill.

So far as I know, little has been done heretofore by the engineering profession in the matter of obtaining accurate data on the temperatures upon which concrete roads have been placed, but here a record of the temperature of the atmosphere and of the wet concrete in place is required for every 100-ft. station on the roads. A record of the amount of cement used in each individual station is also required and furnished from day to day to the office.

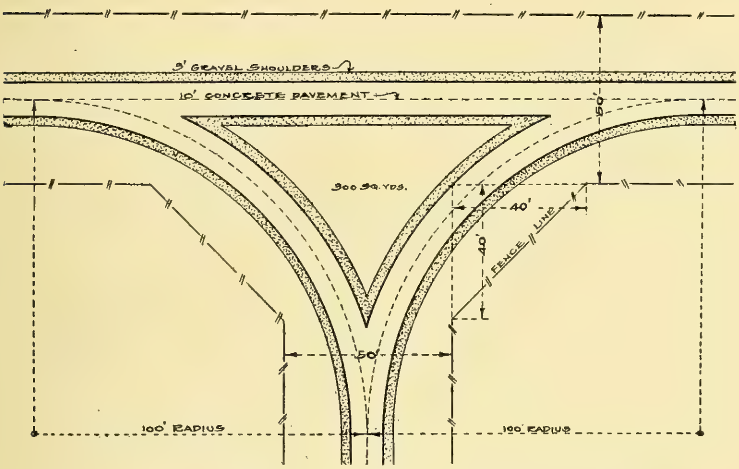

FIG 1.—METHOD OF MAKING A T INTERSECTION.

Cement is tested under a contract with an engineering laboratory after competitive bids. These tests are required to be made on each carload of cement shipped. Samples are taken from cars at the plants and cement is held until preliminary tests are made. The following are the requirements of the contracts for testing cement:

Chemical Tests:

1 Insoluble residue 1 Loss on ignition 1 Magnesia (MgO) 1 Calcium sulphate (CaSO)	one on each tenth car.

Physical Tests:

1 Boiling test from each tenth barrel.
1 Setting test from each tenth barrel.
1 Fineness on a mixture of samples from each carload.
3 Tensile tests neat 7 days
3 " " 1 : 3 7 "
3 " " neat 28 "
3 " " 1 : 3 28 "
on a mixture from each carload.

Test reports are made to the superintending engineer of Vermilion County Bond Roads, Danville, Illinois, and to the State Highway Engineer, Springfield, Illinois. Tests are conducted according to the methods laid down in Circular 33, U. S. Bureau of Standards. Tests are made at the respective plants as set forth in contract with the County of Vermilion.

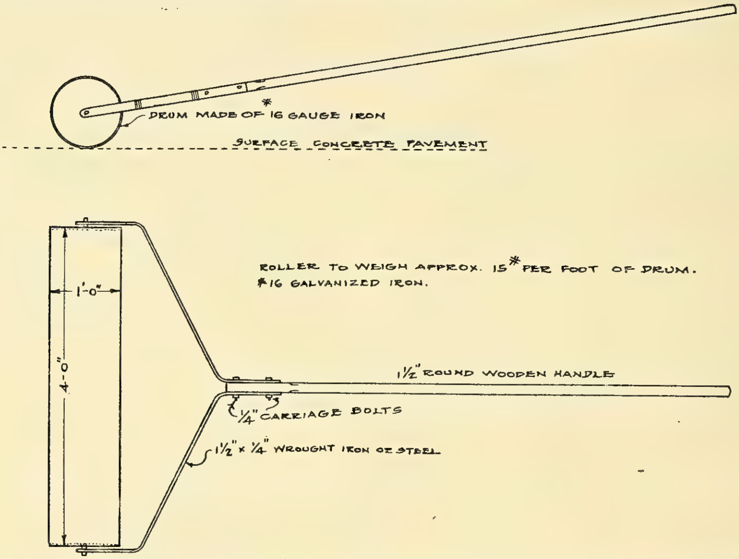
DRUM MADE OF # 16 GAUGE IRON
SURFACE CONCRETE PAVEMENT
ROLLER TO WEIGH APPROX. 15# PER FOOT OF DRUM.
#16 GALVANIZED IRON.
1'-0"
4'-0"
1½" ROUND WOODEN HANDLE
¼" CARRIAGE BOLTS
1½" × ¼" WROUGHT IRON OR STEEL

FIG 2.—STANDARD ROAD ROLLER FOR VERMILION COUNTY CONCRETE ROADS.

A record is also kept and forms are furnished for the purpose of showing portion of slab covered and how much sprinkled each day, and if not, why not.

In addition to the tests made by the testing company division engineers are required to furnish samples for test from every tenth carload received on the division. These samples are placed under test in our own laboratory at Danville to check against the test made by the engineering company. In addition, the local representative of the Illinois State Highway Department is also privileged to take samples of cement, coarse or fine, and send them to Springfield to the state testing department for test and report.

In the placing of concrete road under Section 74 of the contract, which required that the finishing should be done with wood floats, it was found practically an impossibility to prevent the ordinary worker or even the expert finisher from making hollow spaces in the concrete road, and on work where crushed stone aggregate was used it seemed to require too much physical exertion to bring sufficient mortar to the surface. Section 74 is as follows: "After the surface of the concrete roadway has been struck off to the proper cross-section, it shall be finished with wood floats. The wood floats shall be used only to flush mortar to the porous places in the surface, and great care must be taken not to rub hollow places in the surface. The wood float finish shall be made as soon after the concrete has been deposited as possible, and in no case shall the finishing be delayed until the concrete has taken a set."

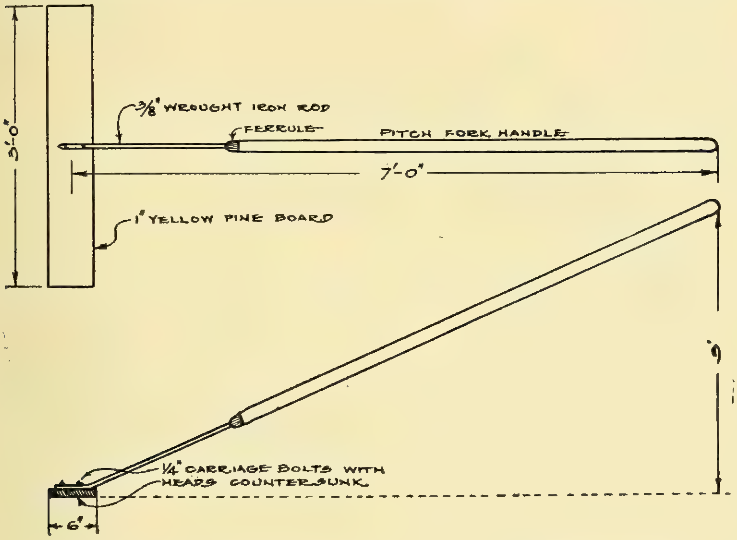

FIG 3.—ONE TYPE OF HAND FLOAT USED IN VERMILION COUNTY.

Appeals to inspectors or to division engineers and injunctions to the contractor failed to make enough impression to get the work done as it ought to be. Accordingly it was early determined to use other devices Our experience indicated that the use of the hand float or the long handle float with a circular rotary motion was an ideal method of making water pockets in the pavement. It was found that the tendency on every job was to add too much water to the mix, and this latter finally determined us to experiment with other devices with a view to eliminating these defects. Accordingly belts were tried in conjunction with the long handle float with the requirement that the long handle float should be used only in a transverse direction across the slab. A combination of the belt and long handle float used in this manner improved the work somewhat, but was not entirely satisfactory.

Accordingly it was determined to try a roller of the type designed by Colonel Gaillard, City Engineer of Macon, Ga. (Details of the float and roller are shown in Figs. 2 to 4.) On our widths of road this type of roller in conjunction with the long handle float proved an immediate and unqualified

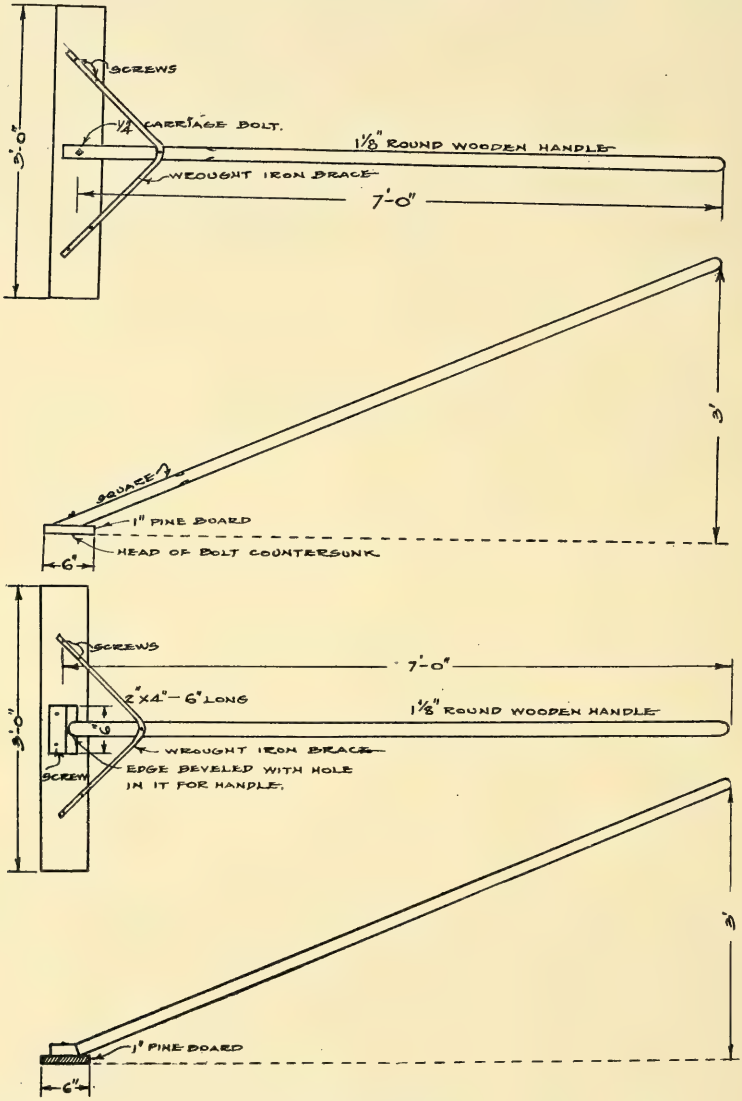

FIG. 4.—TWO TYPES OF HAND FLOATS FOR VERMILION COUNTY CONCRETE ROADS.

success and since that time every one of the contractors has purchased a roller and long handle float for this purpose and although not required to do so by the contract or our specifications. The roller gives better work in many other respects not the least of which is the elimination of hair cracks due to high wind or heat or both, in addition to preventing the hollow spots referred to before, caused by the circular motion of either short or long handle floats.

I have personally driven over miles of road finished with the roller and long handle float immediately after a rain and have not found as much as one depression upon the entire distance.

Our contract in regard to joints in concrete roads requires that "whenever the temperature of the atmosphere exceeds 70° F. joints shall be provided at intervals of 200 ft.; whenever the temperature is less than 70° F. joints shall be provided at intervals of 100 ft."

Whether due to the fact that the concrete placed at the lower temperature was better concrete (all were laid with equal care and supervision), or from the fact that the joints were spaced closer, it is a fact that fewer cracks have occurred in the concrete roads when the joints were placed 100 ft. apart than when placed 200 ft. apart. Placing joints as designed was decidedly empirical as there was no general agreement between engineers but rather a radical difference on the question whether joints should be used at all in rigid types of road.

After considerable experience both in the State Highway Department of Illinois and as superintending engineer of Vermilion County, I have no hesitation now in saying that joints should be placed much more frequently and of a wider type than recommended. The spacing and thickness of joints as shown in this section of our specifications was based only upon the coefficient of expansion of concrete due to heat alone and for a range of 60° F. Expansion due to moisture or crystallization was not considered and while this has not yet been definitely determined there is strong evidence bearing on the subject from several sources that expansion due to moisture content is as great or greater than that due to heat, and accordingly, I believe a sufficient number of joints should be made to prevent any stresses in the slab due to expansion from either moisture or crystallization or expansion due to heat or both. While the concrete slab may within itself be strong enough to resist internal stresses due to expansion once in a while, there can be very little doubt that repeated internal stresses however slight destroy the cohesion and adhesion of the aggregates in a concrete structure if adequate provision is not made for such expansion.

Fifty miles of the concrete roadways of Vermilion County have now been in service two seasons, and although I do not believe the work was nearly as well done as in 1917, they have given very good satisfaction and show small signs of deterioration. Repeated examinations of the roller completed roads, however, show an exceedingly more dense surface.

SUPERELEVATION OF CURVES ON CONCRETE HIGHWAYS.*

By A. N. Johnson.†

In recent discussions of the amount of superelevation that should be given highways at curves the writer has failed to note any mention made of the resistance of the pavement surface to slipping or skidding and how such resistance should be considered in determining what is the proper superelevation. One effect of the failure to consider this phase of the problem has resulted in unnecessary refinements in the practical application of the mathematical formulas involved.

The first consideration in the development of the problem is to decide as to the speed for which the curves are to be made safe, whether 20, 30 or 40 miles an hour. It will be assumed that they are to be made safe for 30 miles an hour. Whether this is the speed that will ultimately prevail or not the mathematical treatment will be the same.

When a motor vehicle rounds a curve there is developed a thrust or push tending to force the vehicle to the outside of the pavement. The only reason that the vehicle does not slide or skid is because of the friction between the wheels and the road surface.

Referring to Fig. 1, there is represented a motor vehicle on a level surface moving on a circle, the point C is the center of gravity of the vehicle and CH represents the horizontal force or thrust that is tending to force the vehicle to one side. The line CP is the resultant of this force with the force of gravity acting through the center of gravity. The angle that the line CP makes with the vertical CA we will call the angle of slide, or angle of friction. It is evident that the greater the force CH the larger the angle of friction, or the angle a as it has been lettered.

We know from mechanics that $\tan a = \frac{v^2}{gr}$ where v is the velocity in feet per second of the vehicle along the curve; g, acceleration due to gravity, equals 32.2, and r is the radius of the curve in feet. It is therefore evident that if we can determine the angle a we shall be able to compute r for a given velocity.

We are not concerned with the ultimate value of a, when the car is just on the point of skidding, but with the highest value which we feel to be safe. This may be determined experimentally in a number of ways; two were made as follows:

In the first case, on a curve marked upon the surface of a concrete pavement, a motorcycle was operated at varying speeds, increasing to a point beyond which the rider felt it was unsafe to go, and the largest angle of inclination that the motorcycle made with the vertical was observed.

In the second instance a board was set up vertically in an automobile and attached to it was a pendulum. As the car was driven around a marked curve the angle made by the pendulum with the vertical was observed. The car was also operated at varying speeds to that which the driver thought was just safe. In this latter method it is to be observed that the angle obtained is a little in excess of the true angle, due to the movement or swing of the body of the car upon the springs. The tangent of a was observed in the case of the motorcycle to be 0.37 and in the case of the pendulum in the automobile to be 0.43. Each of these values was the result of a number of observations.

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* Submitted at Annual Convention as discussion of papers on concrete roads.

† Consulting Highway Engineer, Portland Cement Association, Chicago.

It is evident that a value of the tangent of a of 0.35 can be taken as one of entire safety on a concrete pavement in normal dry condition. With this value for the tangent a and with a speed of 30 miles per hour, r is found equal to 172 ft. Thus, on a concrete pavement with a flat surface and on a curve with a radius not less than 172 ft., a motor vehicle may be driven around the curve at 30 miles an hour without skidding.

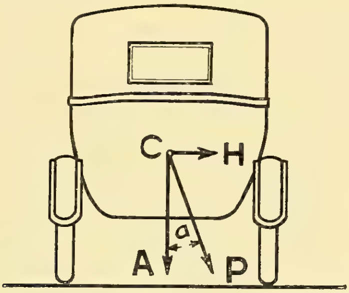

FIG. 1.

We next have to determine the smallest radius around which it may be safe to go at 30 miles an hour with a given superelevation. Before this can be determined the maximum amount of superelevation must be fixed, whether it shall be $\frac{3}{4}$ in. per ft., 1 in. per ft. or as much as 2 in. per ft. Referring to Fig. 2, there is illustrated a motor vehicle on a concrete surface OS which has an inclination with the horizontal OH, represented by the angle i. C, as before, is the center of gravity of the vehicle, CA is the perpendicular, CN is the normal to the surface, while the angle NCP equals a or 19° 17′, the safe angle of friction. If the superelevation is taken as $1\frac{1}{2}$ in. per ft., the angle i equals 7° 11′ and the radius $r = \frac{v^2}{g \tan (a+i)}$ hence r equals 121 ft. That is, a concrete road with a superelevation of $1\frac{1}{2}$ in. per ft. can carry safely motor traffic around a curve with an inside radius of 121 ft. at a speed of 30 miles per hour. If the amount of superelevation is taken at 1 in. per ft. and the other conditions remain the same as here assumed, the radius around which motor vehicles may operate at 30 miles an hour without skidding is found to be 134 ft.

The amount of elevation being so much less than the safe angle of friction there is no need for any special refinement in varying the amount of inclination between the limiting values of r.

If it becomes necessary to construct a concrete road on a curve of less radius than 121 ft., such a curve should have a danger signal warning drivers that it cannot be operated at the regulation speed, here assumed to be 30 miles an hour. In the instance of all other curves it is necessary only that a sign be erected to warn of the existence of the curve ahead but without any danger signal on the sign.

There are considerations other than safety against skidding which require a curve to be superelevated. A natural tendency where a curve is not superelevated but crowned in the usual manner is, on rounding the curve, to cut to the inside when the curve is approached from the outer edge. Such a practice adds greatly to the danger of collisions on curves, particularly where clear vision ahead is confined to a few feet. We will assume that safety under such circumstances require a clear vision around the curve of 500 ft. To compute the radius of the curve it is necessary to determine upon the unobstructed distance from the vehicle to the roadside which will be taken as 15 ft. With these conditions it is found that the radius of such a curve is 2000 ft., so that aside from any precaution against skidding it is recommended that all curves of radii between 172 ft. and 2000 ft. be given a superelevation of $\frac{1}{2}$ in. per ft. We therefore have as a conclusion that highway curves on concrete roads should have the following superelevation: Radii from 120 ft. to 135 ft. $= 1\frac{1}{2}$ in. per ft.; from 135 ft. to 170 ft. $= 1$ in. per ft.; from 170 ft. to 2000 ft. $= \frac{1}{2}$ in. per ft.

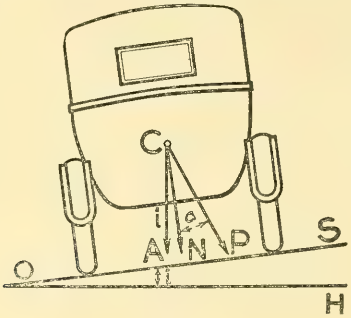

FIG. 2.

Whether the values here assumed are the proper ones or not is open to discussion, but they seem to be both reasonable and practicable. But whatever the limits placed upon the amount of the superelevation or upon the speed, the general treatment would be identical with that here outlined.

If the surface of the pavement is other than concrete the angle of friction should be determined experimentally as was here done, as there doubtless will be found some variation, depending upon the character of the surface. In all cases, however, the several angles should be for the normal condition of the surface and need not take into question an ice-covered or wet surface, for in these conditions all drivers would realize that they must operate their motor vehicles with extra caution.

LABOR TURNOVER AND ITS RELATION TO INDUSTRIAL HOUSING.

By Leslie H. Allen.*

At first sight a discussion of labor turnover at a meeting of the American Concrete Institute may seem rather out of place, but this subject is assuming such national importance, and is having such an effect on every branch of industry, that its interest to members of the Institute will not be hard to find. The country has at last awakened to the fact that there is a very serious shortage of housing facilities for the working class. This is particularly important around munition plants and other industrial centers where factories and mills have been extended without any addition to the housing accommodations around them. The result has been serious overcrowding and general discomfort which in its turn has caused an unprecedented restlessness among the working people and a large addition to the class of "floaters" who will not stay at any one job for a length of time.

The directors of the Concrete Institute, realizing the great importance of the housing question, have appointed a committee, with the writer as chairman, to study the industrial housing question, with particular reference to the use of concrete in the house construction, and it seems fitting to the committee that a discussion of the methods of building concrete houses should be prefaced by a short paper on the reason for the urgency of the present question; as an intelligent understanding of the root causes of the present trouble should indicate the way to approach the problem and to guide us in thinking out a solution.

The most pressing problem of the manufacturer is to obtain and maintain a sufficient supply of labor in his plant. The general experience of manufacturers seems to show that a turnover of 300 per cent per annum is not abnormal. That is to say, to keep 100 men on the payroll it may be necessary to hire three times as many in each year. Many of the men, perhaps half, in many industries are steady men who are not likely to leave and this means that the unstable element is changed about six times in each year. There are many industries who can show a far worse record than this. One public service corporation in Pennsylvania, having reached a turnover of 1000 per cent and even more, is shown occasionally when figures are disclosed.

But it has been found that those plants that have plenty of decent homes around them do not find anything like as much difficulty in holding their men; for this reason employers are giving more and more attention to industrial housing as a means of holding their men.

The provision of housing around the plant has not heretofore been considered the duty of the manufacturer. The local supply has usually been maintained by the local real estate operator and the speculative builders, and the manufacturer considered that when the whistle blew and the workmen left, the manufacturer's responsibility in connection with his help was over. The chief exception to this rule has been in mining towns and other communities which depend on a single industry for employment.

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* Aberthaw Construction Co., Boston, Mass.

The present high prices of material and labor have so raised the cost of house building that it is no longer a remunerative investment for local capital, and money is not available for loan from local banks for construction work. Tenants are uncertain and rents cannot be depended upon to give a high enough return to compensate for the expected fall in values at the close of the war, for these reasons the burden must fall upon either the manufacturer or upon the city or state to provide the houses so urgently needed. The present conditions are almost incredible, overcrowding with six or eight men in a room and two or three shifts occupying the same bed, are now common in any industrial center. Men cannot bring their families with them, they pay exorbitant rates for their lodgings, that are so wretched that we cannot blame them for desiring to move from place to place if only for a change from one set of miserable conditions to another.

This large addition to the class of "floaters" is a most serious industrial problem. Because of this, the manufacturer is losing men, losing work, losing output and losing profit; the reduction in output is likely to have a very serious effect upon the successful waging of the present war. And the workman is no better off; high wages and shorter hours are not bringing any comfort or happiness to him or helping him to establish himself in a good home. Rents rise as fast as wages do, and but few workmen are able to buy, as the financial problem is one that is too much for him. Very few workmen who buy lots are ever able to build houses on them and many lose their holdings owing to their inability to meet assessments for betterments.

The manufacturer is beginning to realize this, the last few months have seen a great interest on his part in housing as an aid to production. The Government also has investigated the subject with the result that $110,000,000 has been appropriated to relieve the situation at important points, where local capital is insufficient. This sum is too small to accomplish much. Ten times that amount is needed if our working classes are all to be decently housed in hygienic homes, amid sanitary and congenial surroundings.

The importance of the new phase of the subject to us is that small house building in the near future will probably be carried out either by manufacturers or out of government or municipal funds.

The manufacturer, building houses for the benefit of his business, is more likely to desire good construction than the speculative builder who desires to sell quickly, it is fairly certain that if the Federal Government authorities decide to contribute to housing developments they will demand a type of construction that will be permanent. The English Government, which has spent millions of dollars in housing since the beginning of the war, has in almost every case put up brick structures and not frame, and it is probable that this example will be followed by the government here in their housing work for the shipping board and in munition centers.

The American Concrete Institute is interested in the use of concrete, but, up to the present, has not given much attention to the small house problem. This committee was appointed with the intention that a proper study and report should be made of the concrete housing work that has been done and the methods of doing it, and the future prospects of work along this line.

It is our belief that the advantages of the concrete house will appeal very strongly to manufacturers, and it is hoped that the impetus given to the building of well-designed houses with good material will be a starting point for a real improvement in housing conditions. The aid of concrete users is needed to gain the greatest good from the movement now started.

A man that owns a comfortable permanent home is not likely to leave it. When others are leaving a plant he will probably remain and be a permanent asset to the community in which he has made his home. A sanitary and hygienic concrete house, designed to admit a proper amount of sunlight and ventilation to every room, should play a great part in improving his health and physique and provide those conditions that are essential to his comfort happiness and prosperity.

The provision of good housing not only meets a local need but it makes healthier families and happier citizens in a more united nation.

ADVANTAGES AND DISADVANTAGES OF THE CONCRETE HOUSE.

By John E. Conzelman.*

It will be necessary to define just what is meant by the term "concrete house" as it is here used. We are accustomed to speak of brick houses, stone houses or frame houses, the distinction being based on the construction used for the exterior walls. Similarly a concrete house could be classified as one with concrete walls. In such a case, the walls might be built of concrete blocks, of concrete poured in place, of unit construction, or of the plastered type of construction.

A house constructed with concrete walls of such character would have much in common with the brick or stone houses and even with the frame houses mentioned. In such a case the question of whether or not it would be advantageous to use concrete walls would resolve itself into a matter of cost, and would be settled by the local conditions governing this particular problem.

It would be impossible to dwell much on the comparative advantages of such a concrete house. There are so many other things entering into the construction of a house of this type which require attention and which affect the maintenance cost that the type of wall used will have little effect on the final analysis.

When we come, however, to a consideration of the house in which the floors as well as the walls are made of concrete we have an entirely different proposition. Such a house cannot be considered as being in the same class with the others; the introduction of concrete floors has put it into the first class. The house has in fact become fireproof and permanent and we realize that if all or most of the houses in a community were so constructed, the fire losses from individual fires as well as those resulting from a general conflagration would be nearly eliminated.

Those who are familiar with the devastation caused by rats, mice, and other vermin that find easy lodgment in the open walls and floors of frame or brick buildings, estimate the loss in property and in the health of the inhabitants at millions of dollars annually. Practically all of this loss could be avoided by constructing vermin-proof concrete buildings.

We are compelled by law, in practically all of our larger cities, to build factories, hotels and apartment houses of fire resisting construction, and in the natural evolution of the thought, will surely be compelled at some time in the not far distant future to build our city houses of the same construction. As a believer in the precast or unit method of construction I can already see in my mind's eye concrete lumber yards rapidly replacing the present "wood lumber yards."

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*Unit Construction Co., St. Louis, Mo.

The concrete house shall therefore for the purposes of this paper be considered to mean a house of fire-resisting construction with concrete walls and reinforced-concrete floors. Unimportant parts of the building, not comprised in the structural elements, may be built of other materials. For example, we would allow the wood floor finish to remain in such a house provided we cannot persuade the owner that a cement floor finish, although apparently hard and cold, is in reality a warm friend because it is sanitary and cheap.

The value of reinforced-concrete construction has been understood and appreciated for many years so far as factories, warehouses, docks and commercial structures are concerned, and the growth of this class of construction has been little short of marvelous. It would indeed be most extraordinary if the same sturdy advantages which have been demonstrated in innumerable commercial structures would not apply equally well to the construction of houses.

The question may naturally arise at this point of the argument to the effect that if the above statements are true, why have more concrete houses not been built? The answer to this question is easily given. Factories and warehouses are built by men who figure the capital invested in connection with insurance rates, maintenance charges, depreciation value and similar items and who realize that the first cost is not always the last cost. Such men realize that by adopting concrete construction they can secure a building with the best fire-resisting and weather and wear-resisting qualities, and that as a result the total capitalized investments will be less at the expiration of five, ten or twenty years than for cheaper forms of construction.

It has not been the custom to build houses on such a businesslike basis—and the first cost was usually all that was thought of. This is especially true of houses built to sell. The situation, however, is rapidly changing. Corporations and individuals are now realizing that the housing accommodations for their employees are to be considered as a legitimate part of their plant outlay. They are therefore giving to the house building problem the detailed study heretofore reserved for their industrial plants.

Under such conditions the advantages of the concrete house will be quickly comprehended and its adoption is a foregone conclusion. A manufacturer who has constructed a colony of concrete houses for his workmen rests secure in the knowledge that no fire can interfere with his production, at least so far as labor is concerned; his investment is safeguarded and if he cannot secure a low rate of insurance he will be justified in carrying it himself.

At the present time there exists in many localities an enormous and pressing demand for houses for working men. It will be necessary to build houses by the hundreds, and in a few instances by the thousands, if the demand is to be met in the proper way. The necessity of safeguarding such large investments, whether the operation be financed by individuals, corporations or the Government, is apparent. For this very thing the concrete house fulfills the requirements of such conditions better than any other type.

We all want to know facts and figures on the upkeep charges and depreciation rates on concrete buildings and also on the more usual forms of construction. It is difficult to obtain reliable data on these subjects.

Mr. Prescott F. Hall, in a paper read at the National Housing Conference, 1916, has the following to say in regard to depreciation:

"A recent canvass of 22 insurance companies, insurance agents, builders, and architects showed in their opinion an average depreciation of 1½ per cent per year for brick and 2½ per cent for wood. Other estimates were 1 to 1½ per cent for brick and 5 per cent for wood. On the cheaper class of wooden buildings, 5 per cent is probably the more correct figure.

"In February, 1915, the Lloyd-Thomas Co., which has just completed an appraisal of the city of Wilmington, Del., amounting to $73,000,000 of assessed values, communicated to the Building Committee of the Chicago City Council the results of their findings as to the depreciation of the various classes of buildings as follows: In the better residence, the average age of the brick was nine years and the depreciation was 4 per cent, there being no depreciation in the first five years and it being 1 per cent yearly after that. In the frame, the average age of the better class residence was fifteen years. The physical depreciation was 20 per cent and the obsolescence 10 per cent. Obsolescence is partly due to going out of style and this is due largely to lack of architectural planning. The total depreciation was 30 per cent. In the middle class residence, the average age of brick and frame were the same, fifteen years, and the total depreciation of the brick was 20 per cent and the frame 45 per cent or two and one-fourth greater in the frame."

You will see that no data is given for concrete construction. However, the rate of depreciation for brick is given at 1½ per cent per year. This is a low rate and we know that the rate for concrete must be considerably less than for brick and wood construction. If it is much less, it will be a negligible factor. We will have to take upkeep, or repair charges into consideration if we are to give concrete its proper due. Here again we must admit that we have practically no reliable data on upkeep charges, but will have to use our reasoning faculties to arrive at any result.

We are told by the cement manufacturers that "concrete is permanent" and those of us who have engaged in making concrete for many years know that this statement is true, provided suitable materials have been used and that these have been mixed long and well and in the right proportions.

Well-burned bricks are also permanent—perhaps not so permanent as concrete, but still sufficiently permanent to be considered so for all practical purposes. It is not in the walls therefore that we will find any great advantage for concrete construction. When we come to the consideration of the exterior woodwork, such as porch floors, railings and posts, to say nothing of steps, the advantage of concrete construction becomes very apparent. It is such items as these that make the upkeep of ordinary construction so high.

In a reinforced-concrete house the floors and vertical supports are rigid and unyielding, and so not shrink. As a result there is little cracking of plastering and settling of floors, resulting in loose trim and sticking doors in a concrete house. These are the items that call for continual repairs and which are eliminated from the upkeep charges of a concrete house. In some types of concrete houses the plastering can be omitted entirely.

In a consideration of depreciation and upkeep charges, it must be borne in mind that certain items which total a large part of the cost are common to both the concrete and ordinary constructed house. I mean such items as plumbing, heating and lighting fixtures, roof covering and hardware; and many others of a similar nature. These items will have practically the same depreciation in a concrete house that will exist in a brick or frame house and must therefore not be considered in a comparison of values.

It has been stated that the average economic life of a house—by which is meant its period of usefullness for the purpose for which it is built—will not exceed 35 or 40 years in most American cities. It is therefore seen that we will have to make some effort to extend this period of economical life if we are to make full use of longer possible life of the concrete house.

Such effort cannot be applied to the individual house but must be a broad movement affecting the entire neighborhood, if not the entire city. For lasting success, city planning and districting must come to the rescue and stabilize conditions and values. It should be possible, by such means, to prolong the economic life to 60 or 80 years, perhaps even indefinitely as has occurred in numerous European cities. If the economic life were doubled, enormous savings to the country would result.

Any discussion on the advantage of fire-resisting construction would be incomplete without taking into account comparative fire insurance rates. Mr. Grosvenor Atterbury, Architect, has given some information on insurance rates on different types of residences now in force in the locality of Forest Hills, Long Island. He states that the ordinary rate in the vicinity of Forest Hills varies from 40 c. per hundred dollars value on brick houses to 60 c. on frame dwellings. The rates given covering a period of three years.

The houses in Forest Hills Gardens, as you no doubt know, were built by the Sage Foundation Homes Co. and this development comprises a group of most modern and substantially built houses. Most of these houses are of special construction with masonry walls and tile roofs. Metal lath is used throughout and all vertical supports are of metal or masonry. On this type of house the rate is 25 c. per hundred. Among these houses are several groups of concrete houses built under Mr. Atterbury's method of precast construction. These houses have cellular walls, reinforced-concrete floors and roofs, and may be classed as really fireproof construction. These houses receive a rate of 20 c. per hundred dollars valuation.

The building codes of practically all cities are so framed that they practically discriminate against this form of construction. Instead of fostering the construction of fireproof houses, these laws have exactly the opposite effect. An example will best illustrate this point. The company with which I am connected is now constructing some concrete houses on Long Island. The floor slabs were of such size and design that a slab 2 in. thick would have been entirely satisfactory from an engineering standpoint. The Building Code required a minimum thickness of 4 in. for all floor slabs. In this particular case the structural concrete is to be topped with a layer of nailing concrete $1\frac{1}{2}$ in. thick, on top of which the wood finish floor is to be nailed. Had these houses been built with wood joist and flooring a single thickness of floor would have been sufficient. However, as the floor slab was concrete, it was decided that in addition to the wood floor just mentioned there would be required $5\frac{1}{2}$ in. of concrete. Such excessive use of materials seriously handicap fireproof construction.

It is obvious that such inconsistencies must be remedied when the properties and advantages of such construction become more generally understood.

There are many of us who believe that houses should be built of standardized sections, just like automobiles. These sections would be made in factories under the most economical conditions of manufacture, and houses would be carried in stock just like other manufactured products. When we get to this stage of progress it will be possible to erect the walls and floors of a house in one day, or at the most, two days, and at a great saving in cost.

Concrete lends itself admirably to such standardized construction and its fire-resisting properties and other features give it distinct advantages over all other building materials for this class of work.

In order to bring the rent or selling price of a house within the means of the poorer paid workingman, it is necessary to avail ourselves of all possible economies in the design and construction. Even if we limit the conveniences to the minimum compatible with proper living, it is practically impossible to bring down the cost of a detached house to a point where such workmen can afford to live in them.

We can meet this condition, however, if we build the houses in groups or terraces; or at least, if we cannot meet it, we know that such construction is the cheapest that can be built. One great objection to building rows of houses of ordinary construction is the fire risk. Another vital objection is the possible spread of vermin. By building of fireproof concrete construction, we can overcome both of these objections and help a long way toward the solution of the housing problem.

It is impossible for any one who has studied the concrete house proposition to dwell very strongly on the disadvantages of this kind of house. The advantages so far outweigh the disadvantages that one is inclined to talk of them and let the disadvantages take care of themselves. And this is eminently the proper course to pursue because the difficulties and disadvantages of the concrete house are all due to the newness of this form of house construction, and will be worked out in actual practice.

We have not yet evolved standard principles of architectural treatment that can be applied to concrete houses and produce pleasing effects. The ordinary architect is familiar with brick and wood construction and uses these materials easily, and in time a special form or treatment for concrete houses will be developed. It is evident that the tendency will be toward simple and straightforward designs because the requirements of the construction demand it. If we develop the architectural treatment along simple, wholesome lines with regard to securing effects by proportions of surfaces, we will have advanced the architecture of the workmen's house considerably.

Untreated concrete surfaces do not as a rule present a pleasing appearance and most of us object to the monotony of such a surface. This condition is aggravated after a rain. It is possible to overcome this disadvantage by some sort of surface treatment without excessive cost.

It is often said that concrete houses are damp and this opinion is generally held by the layman. We all know that the concrete block house was responsible for this thought and that the methods of construction now in use will overcome this trouble. We should not blame any material for defects caused by its misuse, and a well constructed concrete house is as dry as any other.

It is possible to build houses of concrete, houses that are fireproof, vermin proof, entirely comfortable and wholesome, houses that are practically permanent and have a negligible upkeep cost, and it is possible to do this at a price that makes the proposition an economical one from the purely business standpoint.

Why then should we hesitate to adopt such construction? The advantages, even in the present state of the art so far outweigh the disadvantages as to make the latter negligible.

THE CONCRETE INDUSTRIAL HOUSE; A RECORD OF ACHIEVEMENT.

By C. D. Gilbert.*

The genesis of the concrete industrial house is coincident in locality with the early manufacture of portland cement. In 1880 a row of monolithic concrete houses was constructed in Allentown, Pa., for employees of the Coplay Cement Manufacturing Co., under the personal direction of David O. Saylor, the pioneer of cement manufacture. In their simple lines these houses resemble a typical Philadelphia row and could have served well as an object lesson to many builders in more recent years. The houses are still occupied and giving excellent service. Very little maintenance has been required.

In 1892, the Giant Portland Cement Co., constructed ten houses for workmen, likewise in a terrace. They are of monolithic concrete, 1 : 2 : 6 mix, poured in rough board movable forms, 2 ft. high. The walls are 8 in. thick and large stones are freely embedded. They are reported in perfect condition at the present time. (*Cement Age,* May, 1908, p. 440.)

The record of concrete industrial house building appears to show a gap until 1906, when the Lawrence Portland Cement Co., built five two-story, four-room, double houses of concrete block.

About 1905, J. A. Brodie, in charge of refuse destruction work in Liverpool, England, conceived the idea of using the clinker from his furnaces as an aggregate with portland cement, casting the material in large reinforced slabs and constructing low-cost tenements. The building constructed was three stories high, with twelve tenements. Each tenement has a separate doorway reached by a stairway to outside balconies. Each tenement consisted of a living room, two small bedrooms, scullery and toilet. Gas and water were provided. The roof was flat and fitted up as a playground and for clothes drying. *Cement Age* (Jan., 1906, p. 593) says: "The structure of the building is wholly unlike anything attempted in the United States. The ceiling, floor and each of the four sides are separate concrete slabs, weighing one to eleven tons, hauled to the site by a traction engine. The openings for doors, windows and fireplaces were made when the slabs were manufactured and mortices and tenons were left at the edges so that all parts would dovetail. The joints were made with cement mortar and two 1-in. bolts were used at each vertical joint to hold the parts together while the joints were setting. Balconies, stairs, parapet walls and chimney tops were molded block and the chimney and ash-chutes were lined with earthenware pipe."

The same reference contains the evidence that Mr. Brodie experienced trouble with local building codes, as have most pioneers in concrete house building. His plan called for slabs 7 in. thick, whereas the regulations forced him to build them 14 in. thick. This excessive thickness and the consequent difficulties of handling the slabs raised the cost far beyond the estimate and effectually prevented further development of the idea, which was apparently the first effort approaching modern reinforced-unit construction.

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* *Concrete-Cement Age*, Detroit, Mich.

It was in about this period that the concrete block was hailed as the means by which houses were to be built for next to nothing, and block machines appeared in sand banks and back yards in every town.

Writing in 1914 on the development of the concrete house (*Concrete-Cement Age*, April, 1914, p. 168), Rolf R. Newman said: "The date, May 15, 1907, may be taken as the beginning of the ever-growing movement toward the erection of concrete dwelling houses; for on that date the Association of American Portland Cement Manufacturers made its report of the prize winners in the first competition for designs of 'Suburban Dwellings in Concrete.'"

It is certain that the few years following that date saw developed the basis at least of most of the special construction methods that have since been used.

For the purposes of this paper, the concrete industrial house may be broadly defined as: A dwelling or dwellings, detached or semi-detached, in which concrete is the principal structural material, so economically erected as to be rented to, or purchased by, the wage-earner. To the mind of the author the concrete house properly implies at least concrete walls and first floor with a fire-resisting roof. The lines have not been so closey drawn, however, for the examples given in this paper.

Concrete houses may be classified according to construction as follows:

Class 1—Precast unit houses:
  (*a*) Concrete block.
  (*b*) Concrete structural tile.
  (*c*) Large reinforced units.
Class 2—Plastered houses:
  Built of mortar and metal lath in such form that dependence is not placed on other structural material for support of the walls and floors. Mortar may have been placed by hand or machine.
Class 3—Monolithic houses:
  Solid wall { Poured in wood forms.
    Poured in unit steel forms.
  Double wall { Built in wood or metal set forms.
    Built with wall machines.

CONCRETE-BLOCK HOUSES. CLASS 1 (*a*).

For the purposes of this paper it is assumed that the concrete-block house with wood interior is so widely built as not to require detailed description.

Four notable examples are the large groups of concrete-block houses built by Witherbee-Sherman & Co., Inc., at Mineville, N. Y. (*Cement Age*, Sept., 1909), and houses at Ludlow, Mass., built by the Ludlow Associates (*Concrete,* March, 1911, p. 27), and those built in 1913 by the United States Portland Cement Co., Denver, Col. (*Concrete Cement Age*, March, 1913, p. 138), also those built by the Universal Portland Cement Co., at Duluth, Minn. (*Concrete*, Jan., 1918, p. 31).

In Norfolk, England (*Concrete*, Feb., 1913, p. 73), several cottages have been built by the Wayford Tenants, Ltd. They are one and one-half stories high, L-shaped in plan and have five rooms. The walls are built of concrete block 18 x 9 x 4½ in., laid in double walls with a 1-in. air space. The partitions are of the same block placed singly. The floors are of conrete tile and the roof of concrete roofing tile, both of which are reported successful.

The Crayford Cottage Society, under the direction of the Rural Housing Organization, has built several houses (*Concrete*, Jan., 1916) of concrete block. Each dwelling has a front of 24 ft. and a depth of 17 ft. There are two principal rooms on the first floor and two small bedrooms on the second. Designs are plain and in good taste and the cost has been found to be considerably below that of other building materials. Interior partitions are of cinder concrete slabs, 2¾ in. thick. The chimney block and other special units were cast. The first floor is of concrete, the second floor of frame. The blocks are rough-surfaced and stuccoed. Inside walls are plastered direct.

An Oklahoma builder, who prefers to remain anonymous, has constructed a number of low-cost houses in connection with real estate development (*Concrete*, July, 1912, p. 56). He describes them as one-story buildings, 20 x 36 ft. in plan, exclusive of porch. The floors are concrete—there are five rooms. Each house is furnished with cistern and water connections and with a fireplace in the living room. The walls and partitions are of concrete block and there is a steel ceiling under the concrete roof. Some of the houses have flat roofs with wide cornices and low parapets, with outside stairways to make the roof accessible for use.

Other houses were the same in plan but covered with a tile roof, presumably concrete tile. The builder relates the experience of one of the householders, suddenly called away, in whose absence a gasoline stove exploded and completely burned the wood furnishings of the kitchen. Two children were asleep in the house at the time. The fire burned itself out without awaking the children and without material damage except to wood furnishings and trim in the room in which it originated (*Concrete*, Mar., 1913). These houses have been in the path of a cyclone without appreciable damage. Careful tests show a 14 per cent saving of fuel over adjoining frame house of identical plan.

HOUSES OF STRUCTURAL TILE. CLASS 1 (*b*).

The development of structural concrete tile has not figured largely in industrial housing.

Houses at Youngstown, Ohio.—A. A. Pauly of Youngstown, Ohio, conceived the idea of casting concrete units in heated molds and succeeded in turning out, commercially, a concrete structural tile. This tile was used for about 200 small houses for the Modern Homes Co., of Youngstown (*Concrete*, July, 1910), and later for housing government employees in the Canal Zone (*Concrete*, May, 1914, p. 221).

HOUSES OF LARGE REINFORCED UNITS. CLASS 1 (*c*).

Houses of Precast Slabs at Des Moines, Iowa (*Concrete*, Jan., 1916, p. 41).—At Des Moines, Iowa, Rollin G. Ogburn has erected the lower stories of several houses of large precast concrete slabs, cast flat and raised by an ingenious device which enables them to be handled with simple equipment. He has also developed a system of light plaster cores, used in connection with concrete floor construction. His houses while not strictly industrial, seem to hold suggestions.

Concrete Bungalows in California (*Concrete*, Nov., 1914, p. 204).—The community court as a means of saving land and economizing in building operations has perhaps its greatest development in California. Irving J. Gill, architect, has used concrete in building groups of small dwellings with exceptional effectiveness. Mr. Gill's designs cut out all complicated ornaments and depend for their architectural effect on dignity of line and proportion. The houses are built of large slabs cast on the ground with special metal frames embedded and then raised to position in large sections with equipment similar to that developed by Col. Aiken and used in connction with army barrack construction.

Simpson Craft Houses at New Durham, N. J. (*Concrete*, Jan., 1915, p. 26).—A type of unit-built concrete houses developed by John T. Simpson is well adapted to industrial construction. Its distinguishing feature is the comparatively small size of the unit. The wall slabs are precast and given their surface treatment before erection. To erect, the slabs or panels are set up on the foundations and secured in place by concrete studs cast in metal molds, clamped around them. The molds are made so that either a flush surface or raised beam effect is secured. Floors are constructed of precast beams recessed to receive ceiling panels and over them precast floor slabs are laid which are finished as desired. These houses have been built at New Durham, N. J., and elsewhere.

The Russell Sage Foundation Houses (*Cement Age*, Dec., 1910, p. 315).—The year 1910 saw the construction of a group of houses for the Russell Sage Foundation at Seawaren, N. J., from plans worked out by Grosvenor Atterbury. The first houses had a floor area of 18 x 28 ft. and were built of 120 separate precast units. The sections were made hollow and in some of the work are plastered and in others finished so as to be set in place ready for use. Erection was done by a crane especially constructed. Following these initial houses, which were in nature of an experiment, came the extensive work at Forest Hills Gardens, L. I. (*Concrete*, Jan., 1915, p. 3), which is widely known as a model housing development.

A Concrete Unit House in Los Angeles (*Cement Age*, May, 1912, p. 235).—In connection with the study of the housing situation for Los Angeles, about 1910, Thomas Fellows, designed and superintended the erection of a house, 25 x 26 ft., containing four rooms, bathroom and pantry and a concrete terrace.

The foundation was built monolithically in the usual way. The walls and partitions were cast in slabs on the job and set in place with a builder's setter derrick. The wall slabs were 4 in. thick and reinforced. Connections were made at the corners by means of eyebolts and pins which were afterwards grouted into place. A slightly pitched unit-slab concrete roof was used but the first floor was of frame and wooden cornice was used.

The cost of labor and material was 10 per cent greater than a frame house of the same size and design. The climate of Southern California is such that it was considered unnecessary to provide insulation over the four-inch wall.

One Hundred and Forty-six Unit Built Concrete Houses for Youngstown Sheet and Tube Company (*Concrete*, Jan., 1918, p. 5).—There are now being built by the Unit Construction Co., St. Louis, for the Youngstown Sheet and Tube Co., Youngstown, Ohio, a group of 146 concrete houses.

The houses are built in small groups or terraces with apartments to accommodate from one to three families. The apartments consist of four rooms and bath. For each single apartment, less than 60 separate precast pieces of concrete are used. The entire wall for a story height is in one piece as is the floor slab of each room. Wall slabs are cast with ribs arranged to receive furring and lathing. The second floor ceiling is a concrete slab but the roof is tile on frame construction. The units are handled at the casting yard by a locomotive crane, transported by a special motor truck and set by a special traveling crane.

MONOLITHIC CONCRETE HOUSES.

Popular interest in concrete as a house-building material may be said to date from the announcement by Thomas A. Edison in 1908 that a workman's house could be poured complete as a monolith and sold for about $1200.

In spite of the fact that Mr. Edison's dream of the low-priced industrial house has not materialized, any review of concrete industrial house construction would be incomplete without reference to his idea. He has given the concrete house much publicity—he has also given the house builder a tremendous handicap because of false ideas of cheapness raised in the public mind. In an authorized statement (*Cement Age*, March, 1908, p. 268) to E. S. Larned and Percy S. Wilson, Mr. Edison proposed that the house should be 21 x 45 and 35 ft. high, inclusive of the cellar; walls to be 12 in. thick, reducing to 8 in., roofs 6 in. thick; floors and partitions 4 in. thick. He proposed to construct these dwellings on gravel soil where the aggregate would be very cheap. Molds would consist of cast iron plates, variously estimated to weigh from 140 to 225 tons per house. Mr. Edison's estimate of the cost per house was $1200, to include plumbing, heating and lighting. A year later (*Cement Age*, July, 1909, p. 50) reference was made to a modified plan and to the fact that the new house was designed by George A. Small and Henry J. Harms, Jr., engineers engaged by Mr. Edison to prepare the molds, make the necessary experiments, etc. That the work of these men has in a limited degree justified Mr. Edison's early predictions has been shown by recent work.

Messrs. Harms and Small, convinced of the possibilities that lay in unit molds for concrete houses and not securing adequate support in America, transferred their work to Holland, and on May 3, 1911, they poured a small six-room house complete in six hours (*Cement Age*, Feb., 1912, p. 63). The molds are of very light cast-iron units, connected through slots at the edges and held in position by separators and bolts. A distinguishing feature is that the plates are tied through the wall by bolts through their middle, rather than at the edges of the plates. The difficulties of pouring a house complete in one operation are: first, that of securing a concrete that will flow into all the thin wall and floor sections; second, the necessity of exceptionally rigid forms to stand the pressure of continuously poured work; and third, the necessity of forming floors on the top as well as bottom to prevent concrete rising above the floor level as walls are poured; fourth, the necessity of tying up forms for supporting horizontal sections an excessive length of time.

The first difficulty was successfully overcome by adding a colloidal material, in this instance clay, to the concrete, and by very careful proportioning and exceptionally thorough mixing. A homogeneous, non-segregating fluid mix has been successfully used. Subsequent study developed a composite system using precast hollow beams for floors, roofs and stairs, in connection with monolithic walls (*Concrete-Cement Age*, May, 1913, p. 209). The work has progressed steadily and several industrial developments of considerable magnitude have been carried out in France (*Concrete*, Jan., Feb. and Mar., 1918).

Houses at Rosslyn Farms near Pittsburgh (*Concrete*, June, 1910; *Concrete*, April, 1911, p. 27; *Concrete-Cement Age*, December, 1912, p. 57).—W. H. Parrish, a Carnegie, Pa., builder, about 1910, developed a method of combination wood and concrete construction. While the houses built by Mr. Parrish are not, strictly speaking, industrial, his methods are suggestive. He erects on the foundation walls the floor joists and studding as for a frame house, over which he places metal fabric. He then erects forms consisting of 2 x 6-in. uprights and $\frac{7}{8}$-in. boards, 3-in. outside the studs. Between the studs on the inside he sets vertically panels made of 2-in. planks with their outside surface flush with the face of the studs. The concrete is then poured which results in a 5-in. wall, reinforced with heavy fabric and with furring strips projecting 2 in. in the proper position, to receive laths and plaster. The exterior is given suitable surface treatment varied as desired.

Houses at Virginia Highlands, D. C. (*Concrete*, Dec., 1910, p. 27, and *Cement Age*, Dec., 1911, p. 242).—A number of small and attractive houses were constructed in the vicinity of Washington, about 1910. Several types of houses were built, most of them containing six small rooms. In some of them the floors were of frame construction and in others concrete slabs were used. The walls were 6 in. thick with a layer of roofing paper in the center, which, it is said provides sufficient insulation for the range of temperature encountered in Washington.

The unit forms used were developed by Milton Dana Morrill, of pressed steel plates. Their distinguishing feature is an ingenious device by which they are used in two tiers, one tier being swung up above the other without complete wrecking of the form. Houses of similar construction were built near Chicago, for the E. A. Cummings Co. (*Cement Age,* Sept., 1911). Mr. Morrill's subsequent work in housing at Nanticoke, Pa., and elsewhere lies outside the scope of this paper.

Another development of the same period is that worked out by Wm. Dean Hamm, Muskogee, Okla. Mr. Hamm's forms are of steel and comparatively heavy unflanged plates are used. They fit accurately together and are supported by ties and steel studs. A distinguishing feature is that they are perforated at ½-in. centers in both directions with small holes which are filled with a plastic compound. In use the concrete could be placed very wet and the excess water filtered away through the series of holes, with the result that all the cement was retained and excess water removed. The perforations also enabled the plates to be bent to any desired form and straightened again without difficulty. A number of comparatively inexpensive houses were built.

Houses at Lenni, Pa. (*Cement Age,* May, 1911, p. 247).—The development of the concrete house owes much to C. R. Knapp, who built numerous notable residences in the vicinity of Philadelphia. For the development of suburban tracts, a number of bungalows and double houses were designed by Oliver Randolph Parry and built by the C. R. Knapp Co.

Mr. Knapp's method was to use double wood forms consisting of substantial panels which were raised one above the other as the work progressed. Insulation thought sufficient for the climate of Philadelphia was provided by inserting three-ply roofing paper in the walls 2 inches from the outer surface. The walls were 6 inches thick, reinforced with triangular mesh or with ⅜-in. rods in each direction. The exterior surface was treated with a wash 1 : 3 white cement and white sand.

An Economical Oklahoma Cottage (*Concrete,* May, 1912, p. 41).—A monolithic concrete cottage, containing five rooms was built by A. J. Mossman about 1911, in Ada, Okla. He stated that the cost was about $100 more than the same house would have cost in frame. The walls were insulated by a continuous air space and plastered direct.

Concrete Industrial Houses in England (*Concrete,* Mar., 1913, p. 134, and *Concrete,* April, 1914, p. 185).—In York, England, Rowntree & Co. built an experimental concrete cottage in 1912. It is 27 x 30 ft. in plan, contains five rooms and is covered with a reinforced-concrete roof with a suspended ceiling. The walls are monolithic 9 in. thick and the builder comments that he would have made them hollow except for building restrictions; which again emphasizes the need for building code revision as related to the use of concrete in dwellings and other light structures. The roof slab is 4 in. thick and since some of the spans are too great for 4-in. slabs the concrete was molded into ribs standing above the roof surface which was done at less cost than if form work for beams under the ceiling had been constructed.

Concrete Houses at Midland, Pa. (*Concrete,* April, 1914, p. 159).—In developing the unit steel form idea, most inventors have used a series of rectangular plates. A variation of this idea was developed by F. D. Lambie. Four houses were built at Midland, Pa., in 1913. They are 29 x 18 x 27 ft.

from grade to cornice. The walls are solid, 9 in. thick. The Lambie forms consist of a series of light steel channels suitably notched to receive horizontal stiffening members. The forms are supported on belt courses at each story, a distinguishing feature. The houses at Midland have reinforced-concrete floors and flat roofs. Concrete, a 1 : 2 : 4 mix, was placed from a light tower with spouting equipment.

It is regrettable that these houses, as built, proved a disappointment, in that no insulation was provided and condensation became so serious a matter that it was necessary to fur out and lath the exterior walls after the building had been completed and occupied. This experience resulted in making provision for embedding furring strips to which lath and plaster are secured in other houses built by this method.

Houses at Newark, Ohio (*Concrete*, Jan., 1916, p. 26).—While, strictly speaking, not industrial, a group of houses built by Eli Hull, at Newark, Ohio, using Lambie forms, are interesting. Houses contain eight rooms and bath and are fitted with all modern improvements. Six-inch reinforced-concrete walls and partitions were built and the exterior walls were coated with a light dash of stucco. The floors and pitched roof are of frame. Further developments of this method, notably at Newington, Conn., near Hartford, and the work at Donora, Pa., come outside the scope of this paper.

Concrete Cottages at Euclid Beach, Cleveland (*Concrete*, Nov., 1914, p. 222, and Jan., 1916, p. 31).—Another development that is worth study are the camp cottages built by D. S. Humphrey at Euclid Beach, Cleveland. These cottages are intended for summer use only and replace combination wood and canvas summer shelters previously used. The houses, including porch, are about 24 x 28 ft.; there are two rooms and bath. The walls of the house proper are solid, 4½ in. thick, while the wide porch which comes under the main roof is supported by columns. The roofs are flat.

Construction methods are of unusual interest. Steel forms developed by the Hydraulic Pressed Steel Co., were used and the houses poured practically complete in one piece. Concrete was handled by a conveyor devised by Mr. Humphrey consisting of an inclined trough in which a chain equipped with scrapers operates at fairly high speed. This has proven very satisfactory. A unique feature of these houses is the fact that the roofs are covered with soil several inches deep and then sodded. This renders the houses very cool in summer at slight expense and is an idea worthy of further development since a fill of earth would do much to prevent temperature cracks in monolithic roof slabs.

Houses at Rochester, N. Y. (*Concrete*, April, 1911, p. 78).—In 1909 and 1910 several men, working independently, conceived the idea of building concrete walls using a small open-ended box in which the concrete could be rammed, and the box released and moved ahead. One of them was W. N. Britton, a Rochester real estate man, who owned a large tract of land near the Eastman Kodak Works, on which was a plentiful supply of gravel and where he was building a large number of houses for rent and for sale to Eastman employees. Out of crude experiments he developed a machine, consisting essentially of four plates comprising outer forms and a core, which by a movement of a lever could be released at will. This contrivance was started on the wall at a corner, rammed full of a semi-wet mixture of concrete, the lever released and the machine moved forward, working around the building. It was found that under ordinary conditions the concrete would set enough to enable the operator to keep at work continuously. Mr. Britton began by building a concrete cellar and finally ventured to construct entire houses. The Britton houses are about 25 x 27 ft. in plan and contain seven and eight rooms. They have pitched roofs. In the early houses there were two 4-in. walls separated by a 4-in. air space, the two walls being reinforced by heavy wire and tied across the air space with heavy metal ties. After the first experimental work was completed, the concrete work of the house was generally put up in 14 days. The floors are of wood and placed as the building was constructed to act as staging. The exteriors were stuccoed and the interior walls plastered direct. Four to five men constituted the concrete gang. The equipment was improved to meet commercial conditions and soon passed into the hands of the Van Guilder Double Wall Company, Inc., of Rochester, N. Y. Mr. Britton continued building until he now owns or has sold about 200 concrete cottages, mostly to Eastman Kodak Company employees.

Houses at Walpole, Mass.—The Walpole Rubber Co., of Walpole, Mass., built two houses in 1912 which were the first houses built with wall machines in New England. The houses are 25 x 27 ft. with front and rear porches. There are three principal rooms and a pantry on the first floor and three bedrooms and bath on the second floor and a full basement. The pitched roofs were shingled. The exteriors were stuccoed and the interior walls furred and lathed as the architect was doubtful that a concrete wall alone would be satisfactory. The mix was half cinders and half sand mixed with cement, 1 : 6. The houses were provided with electric lights, gas ranges, furnaces and laundry trays.

Double Wall Houses at Turtle Creek, Pa.—At Turtle Creek, Pa., Christy and Campbell, builders, have constructed a number of small houses with the wall machines. One house, built about 1913, has concrete floors and a pitched concrete roof placed on metal lath over very light steel trusses. The basement floor contains the furnace room and a kitchen and dining room, the street floor contains the living room and two bedrooms. Electric lights, plumbing and hot water heat was installed.

Houses for the Canada Portland Cement Co.—There have been built for the Canada Portland Cement Co., near Montreal, a group of concrete houses with double walls. These were built with what is known as the Roughsedge system, using plank forms in connection with special devices for holding them in position and special collapsible cores for forming the air space. They are understood to have worked out economically. The interiors are of frame and they have a shingle roof.

Houses Built of Pneumatically Placed Concrete (*Concrete,* Jan., 1918, p. 33).—Excluding house construction where a wood frame has been covered with gunite, there are now several instances where concrete houses have been built of air-placed concrete.

Some cottages and barracks were recently constructed for the Traylor Shipbuilding Co., at Cornwells, Pa., that are good examples. To erect these walls wood unit forms were erected, leaving a space between them. Over this frame was placed heavy building paper and heavy wire mesh while rods were placed in the spaces between panels. The building was then "shot," the gunite entering the spaces between panels and forming reinforced-concrete studs.

A method of using gunite in connection with paper cores has been developed by J. C. Hain, representative of the Cement-Gun Co., on the Pacific Coast. Very light frame works of wood strips are made and covered with heavy tar paper. They are erected to serve as cores for the wall, the gunite being shot around them and through spaces left between them to form the supporting members in connection with very light walls.

METHODS OF CONSTRUCTING CONCRETE HOUSES.

By K. H. Talbot.*

This paper will deal with the methods of construction, including a general résumé of the types of forms which have been used in solving the housing problem.

Concrete houses may be divided into the following general classification. (1), monolithic; (2), precast; (3), block. The monolithic concrete house may or may not have reinforced-concrete floors, roof and partitions, but is a residence constructed by casting concrete in place between forms. The precast unit house is made up of a number of standardized concrete units, usually manufactured at a central plant and carried to the job by proper handling equipment. Concrete floors, roof, and bearing partitions have been used extensively in this construction. The concrete block is a unit of construction which is sufficiently small to be handled by manual labor instead of by mechanical equipment. Furthermore, the units are standardized so that a number of them placed in the wall can be used for any size of house, provided the design is made to conform in overall dimensions to the size of the block used.

The same general design should be followed out regardless of the type accepted. It will therefore be assumed that the houses are to be built with large flat exterior surfaces without gingerbread, designed to accentuate the solidarity of the building material used.

The history of concrete forms for residence construction is the history of the concrete foundation. Here weight of forms was not a large factor and ultimate economy was offset by the low cost of wood. The lumber was often used on the upper portions of the house. It remained for the large industrial developments to standardize the construction of concrete foundation by the use of standardized forms, and equipment of a size which would allow their erection in a short time.

Reference should be made at this time to the forms designed by Stone & Webster for the Carnegie Steel Co. in connection with 250 houses built at McDonald, Ohio. These were made up of 1-in. lumber with 2 x 4-in. studs as stiffeners. Standard panels 2, 4 and 6 ft. by 8 ft. high were built with studs bored to allow bolts to be slipped into place to tie the panels together. The use of wire ties eliminated much of the bracing which would otherwise have been necessary. While this method of construction is admirable for foundations, and may be even cheaper than steel forms, yet the weight and clumsiness of sections are such that their application to upper story walls of concrete houses is out of the question.

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* Universal Portland Cement Co., Pittsburgh, Pa.

TYPES OF FORMS.

Various manufacturers attacked the problem of a proper wall form design. Among some of the more successful might be mentioned the Van Guilder, the Hydraulic Pressed Steel, the Schub, the Reichert, the Harms, the Morrill, the Blaw, and the Lambie.

The Van Guilder form, or the Van Guilder double wall machine, as it is generally known, consists basicly of four vertical plates 5 ft. long by 12 or 18 in. high, held in the proper position by yokes passing over the top, the yokes in turn being connected to a lever which controls the relative location of the plates. The design is so made that either or both of the walls may be from 3 to 8 in. thick. At the top of each 18-in. course, horizontal reinforcing steel and wire ties are inserted, tying the two portions of the wall together. The character of the work done with this form is of a quality that would recommend its use where speed is not the determining factor. The fact that the form is released as soon as the concrete is tamped into place automatically determines the consistency of concrete necessary and the speed which can be made.

The Schub, Reichert, Morrill and Blaw forms, each have as the basis of design small plates, usually about 2 ft. square around the perimeter of which angles are riveted. Each system has its own method of connecting plates and holding them in proper alignment.

The Morrill form has been used successfully on operations at Nanticoke, Pa., and Virginia Highlands, Va., by casting the wall in horizontal layers at the rate of about 2 ft. a day. Two sets of forms are hinged so that as soon as the concrete in the upper set has become sufficiently hard, the lower set is swung into place, and after being properly spaced is filled with concrete. These forms have also been used to cast houses in story heights by setting up a sufficient number.

The Blaw light wall form has for its basis a plate 2 ft. square, which is held by clips passing through the outstanding legs of the angles surrounding the plate and is lined by angles or channels. This company recommends that for industrial housing where these plates are used that they be built up into sections 2 x 8 ft., held with two stiffener angles. Sections of this size can be readily handled. They also market a concrete house form, which is made from a plate 2 ft. wide and 8 ft. long with proper stiffening angles.

Of entirely different design is the form manufactured by the Hydraulic Pressed Steel Co., Cleveland, Ohio, which consists of steel uprights accurately spaced 3 ft. 3 in. on centers by steel liners. Between these uprights are clamped steel plates with wood backing, the edges being protected and stiffened by steel channels onto which the clamps fit when secured. The uprights are provided with keynote slots, 6 in. on center, for keying and wedging the clamps. These are furthr provided with holes for wall ties 3 ft. apart. These ties consist of rods with tie clamps adjusted to any thickness of wall and serving also as separators. The uprights may be spliced together with steel splices and carried to any height.

The steel liners which space and align the uprights are secured into the keyhole slots of the uprights by keys and wedges attached to the liner. Corners are formed by means of special inside and outside uprights with end plates of special width, according to the length of the wall. The clamps may be turned in keyhole slots to admit the plates after the uprights and liners are erected and the plates may be removed in the same way, as soon as the concrete is sufficiently set, leaving the uprights to support the wall. This method has been successfully used by D. S. Humphrey at Euclid Beach Park, Cleveland, Ohio, in the construction of summer cottages.

The form manufactured by the Reichert Manufacturing Co. is based on a 24-in. square unit. Special units of dimensions graded down to every two inches to a 2-in. plate permit the construction of walls of any multiple of 2 in. in length and one special 3-in. plate used in conjunction with these enables wall to be built any odd number of inches in length.

The unit consists of a galvanized iron sheet with a 1 x 1-in. angle riveted to each side, and an additional angle across the middle of the plate as a stiffener. The top and bottom angles are provided with holes opposite each other, in which spikes are dropped to maintain vertical alignment. On each of the side angles there are small pins projecting $\frac{1}{8}$ in. near the top and bottom which fit into corresponding holes in the angle of the adjoining unit, thus maintaining horizontal alignment. One of the side angles has riveted to it two malleable iron clamps which hold the sides of adjoining units securely together. These units are also wired in a manner similar to wood forms, two holes being provided at each side for that purpose and stay rods, adjustable to the thickness of the wall, hold the plates apart.

A 1-in. angle with clamps riveted to it is provided for outside corners of the wall. The inside corner connection is a 2-in. angle having smaller angles riveted to the extremities of the arms. These have clamps corresponding to those on the adjoining standard units. A hinged corner with clamps can be used for non-rectangular buildings or bay windows.

The Lambie Concrete House Corporation, Boston, Mass., have put on the market a steel form which was used in the construction of the houses at Donora, Pa. These forms consist of units of steel channel units of uniform length, depending on the height of the wall to be built, with an angle riveted across each end. The channels are provided with holes in the flanges to enable keying and wedging to adjoining channels and the angles at the ends are provided with holes for keying and wedging to a special arrangement of channels or angles at the top and bottom of the wall. Grooves are provided near the top and bottom of the units into which is fitted a steel angle which secures perfect alignment. The units are made in two standard widths of 9 and 6 in., thus providing for any wall of a multiple of 3 in. in length.

The Harms form has as a basis of design the use of small plates, but in this case they are made of cast iron instead of steel. The same general method of holding them together applies as used for other small plate forms.

PRACTICAL DETAILS OF USING FORMS.

The experience with steel forms would tend to show that greatest economy can be obtained by concreting the house in one-story lifts, sufficient forms being available to construct one story at a time. Door openings may be either formed around, or frame bucks set in the wall, to which are attached strips which hold the door frames in place. The use of bucks and the carrying of the steel forms past window openings has been found the most economical.

Regardless of the type of forms used, floors should be cast at the same time as the walls of the story below. Economy of construction requires the use of floor forms as supports for men while distributing the concrete for the walls.

The difference in the cost of reinforced-concrete floors and wood floors at this time is so small that I question the justifiability of putting wood floors in such construction. If wood floors are used, the joists should be placed at the time that the forming is done and in order that they may be used as braces, and to support the workmen as the house progresses. For a house 20 by 30 ft. in plan it is probable that the difference in cost would not be more than $100 per floor. To obtain that security of investment that is available only from fireproof construction, the expenditure of this amount is justified.

Economy requires that reinforcing metal in the wall be placed while forming is in progress. The usual method is to place the inside form, then the reinforcement, and follow with the outside form. This method may be reversed in erection of the basement forms. However, as a rule, the basement walls are not reinforced. The floor reinforcement must extend into the walls a sufficient distance to develop the proper bond. To meet the usual design requirement of 40 diameters embedment for bond, it will be necessary to bend down the floor bars in to thin wall construction.

Experience has shown that reinforced-concrete walls from 4 to 6 in. in thickness are sufficiently heavy for residential use, instead of the present 8 or 10-in. requirement of many building codes. It is my opinion that a 4-in. wall properly reinforced and tied to the floor level by reinforced-concrete floors is a superior construction to an 8-in. brick wall with wood joist floors.

METHODS OF PLACING CONCRETE.

Three methods of placing concrete have been used in the construction of monolithic houses. The first consists of elevating the concrete in buggies or wheelbarrows; the second in the erection of a tower from which the concrete is chuted into place; and the third is the system designed by D. S. Humphrey and used in the placing of concrete in his own houses at Euclid Beach Park, Cleveland, Ohio. This consists of a trough with semi-circular bottom, up which travels a series of flights on a chain, the concrete being placed in a hopper at the bottom of the trough, is elevated to the top, and distributed through chutes. The chute need not be very long, as the spout is designed as a swinging boom and is easily moved.

There are on the market small stiff-leg derricks which can be set on the corner of a house, and by which wheelbarrows or concrete carts can easily be hoisted.

When it is realized that a house, including foundation, requires not more than 100 cu. yd. of concrete, it will be seen that equipment for handling concrete must be small, easily moved, and inexpensive, and that the erection of a large tower, from which at best it would be possible to chute only four or six houses is not economical.

A small mixer with a capacity of from 30 to 50 yd. per day, a light stiff-leg derrick and a portable hoisting engine, preferably mounted on self-propelled trucks will give the most economiczl method of placing concrete. There is obviously little advantage in having a mixing equipment which will place the concrete in more than a single story in a working day. If sufficient forms are available so that concrete is to be placed in more than one location in a day, it will be found preferable to obtain other mixing units than to double the size of the unit used.

The method of finishing the surface of the concrete will determine the quantity of spading required, and the care with which the concrete must be placed in the forms. In case it is expected to finish the surface either by floating, wire-brushing, or tooling, great care must be taken to obtain a uniform texture throughout the wall. The form must be properly oiled with a colorless material, care being taken that the oiling is done uniformly so that there will be no variation in color; the concrete must be of uniform consistency, thoroughly mixed, and carefully spaded.

It is of utmost importance that the filling of the forms be carried on in comparatively small lifts, so that the layers will be placed horizontally and no water pockets will form. Otherwise, there will be sand streaks and variations in the quality of the wall surface, and in spots difficulty will be found by the appearance of laitance.

If it is planned to stucco the exterior, it will be found advisable to refrain from oiling the forms, to dry up the mix slightly, to compact it sufficiently so that there will be no large voids in the wall, but to leave the surface rather rough by reducing the spading to a minimum. If this is done more stucco will be required than if the surface were left smooth, but the added bond is worth the cost of the additional stucco.

Forms should be allowed to remain in place for as short a time as possible. This statement is the opposite of the one to which we are accustomed, but I am convinced that many houses will be built in the near future on which no stucco will be applied, but the surface of which will be treated by wire-brushing or tooling. Mr. Humphrey has shown conclusively at Cleveland that a very pleasing finish can be obtained by the use of wire brush on concrete about 24 hours old.

Naturally the forms on floors should be left in place for a longer time than on the walls.

Methods of Finishing Surface.

A discussion of the methods of finishing the surface of the building may not be amiss at this time. Reference has already been made to three general classifications: (1), stucco; (2), wire-brushing; and (3), tooling.

Stucco has been used extensively and very satisfactorily, and has the advantage of its availability in varying colors. However, the experience of such product manufacturers as the Hydro-Stone Co. and the Zagelmeyer Co. leads me to believe that we can get happy color schemes by using the colored aggregates. For instance, in the vicinity of Pittsburgh, we have available a good river pebble, limestone, and crushed blast furnace slag. A combination of these with portland cement and river sand, when the aggregate is exposed, gives a very satisfactory surface. The addition of other materials, such as granite, spar, etc., can be done without greatly increasing the cost.

Two general types of stucco have been used. The first is known as patent stucco, and the second as cement stucco. The patent stucco, so called, is a manufactured article, usually having a lime or magnesium base. The cement stucco is usually made on the ground and consists of portland cement and aggregate with the possible addition of a small amount of hydrated lime.

In applying the stucco, it is essential that the concrete wall upon which it is applied be thoroughly moistened and the work protected against too rapid drying out.

The surface of many engineering structures, notably the Larimer Avenue Bridge in Pittsburgh, has been tooled, bringing out the character of the aggregate. Tooling of the whole surface of a house is expensive, and it is evident that in this type of construction where it is possible to remove wall forms early, it will be found more economical to expose the aggregate by wire-brushing.

Standardization is the basis on which the cast concrete house is to be a success, but this standardization does not mean that each house shall be just like the other, but rather that the surface and overall dimensions shall be such that the same forms can be used throughout the whole operation. The pictures of the concrete houses at Donora do not do the houses justice, in fact, a personal study of the project is required before the beauties of the houses can be appreciated. With forms which can be used successfully to meet any dimension within 3 to 8 ft., such as are manufactured by all of the form manufacturers today, there is no reason why full sway cannot be given to the architect who concentrates on large flat surfaces.

The second type of construction, usually known as precast or unit construction, or as the shop manufactured house, has been worked out by Grosvenor Atterbury of New York City, and John T. Simpson, an architect of Newark, N. J., also by the Unit Construction Co. of St. Louis, Mo. Basicly the methods of these three concerns are the same, although the systems of putting together the units differ.

The Simpson-Craft method consists of the use of a number of standardized units, which after being placed to proper lines are held in position by field cast connections. This applies not only to the wall slabs but as well to floor beams and floor slabs.

In the system designed by Grosvenor Atterbury, as well as that used by the Unit Construction Co. in the construction of the houses for the Youngstown Sheet & Tube Co., Youngstown, Ohio, the slabs are designed so that they fit together without casting in the field. The units in this case weigh from one to twenty tons, as against a maximum weight of 750 lb. in the Simpson-Craft construction.

The principal difference between the two methods is that the wall slab designed by the Unit Construction Co. carries the floor loads directly to the foundation; while in the Simpson-Craft houses, the floor loads are transmitted through beams to girders and from them through columns and pilasters to the footings. This whole system makes possible the use of $1\frac{1}{2}$ and 2-in. slabs. The minimum thickness of wall of the Unit Construction Co. method is usually 6 or 8 inches.

The casting of sections at a central plant offers to the artistically inclined architect and engineer an opportunity to adopt color schemes which are not applicable to the monolithic house without the use of stucco, as in this instance, the slabs can be made either face up or face down upon prepared bed plates. The Unit Construction Co. has found it advisable to cast these sections on concrete forms in order that there may be no warping. Splices between sections are made by splicing the ends of reinforcing steel or by dowels fitted in sleeves in the concrete.

With proper equipment there should be no difficulty in the erection of houses at the rate of one story a day after the units are cast, and if the number of houses make economical rapid moving equipment, it should be possible to erect one house a day with each unit.

This method of construction requires that the units be designed to fit the design of the house, and although it is possible to construct bay windows and special features, the added expense is not justified.

The actual cost of the precast unit method as opposed to the cost of the monolithic concrete house will depend upon the local conditions. Maximum economy is to be obtained by the construction of a large number of houses at one site. For industrial developments that take advantage of scattered vacant properties in already built-up sections of the city, the concrete block offers an economical construction.

Concrete blocks have not in the past been looked upon with favor by architects and by many engineers because of the method of manufacture in the past and the general appearance of the block. However, this objection has been done away with by the careful study of the concrete block, which is basicly a good building unit.

The houses built by the Minnesota Steel Co. at Morgan Park, Duluth, Minn., have proved without a doubt that it is possible to use concrete block successfully in a large housing operation. The use of comparatively wet mixture with varied colored facing, such as granites, spar, etc., has proven very successful.

Personally I feel that the concrete block made in the ordinary machine and with a smooth face laid up in the wall with narrow unpointed joints and with a stuccoed surface is a very economical construction and one which will fulfill the needs in many places. The use of the high grade specially faced blocks without stucco is to be recommended for the very finest of residential construction. The equipment cost for concrete block is low and the number made by machine is such that it is entirely feasible to erect houses rapidly. In the construction of block houses it is desirable that the minimum thickness of bearing walls be 8 in. and that the block meet the specifications of this Institute.

Cinder concrete blocks and cinder concrete in general has been used successfully in solving the housing question. Their small weight, the availability of good cinders in most communities at reasonable prices, and their non-conductivity of air recommends them for such use. It goes without saying that any such construction should be stuccoed on the outside.

I do not believe that we have reached the ultimate economy in concrete house construction, but I do feel that the concrete houses as represented by the operation at Morgan Park, Duluth, Minnesota, Donora, Pennsylvania, and East Youngstown, Ohio, have solved for all time the industrial housing problem.

ARCHITECTURAL DESIGN OF THE CONCRETE HOUSE.

By E. G. Perrot.*

With the growing importance of the necessity for permanent construction of good homes for the wage-earner is the all important one of materials for construction coupled with successful architectural design.

When viewing the successful creations of the small house for the wage-earner which found its early expression in modern industrial villages planned on Garden City lines, financed and built by industrial establishments, in England, notably at Port Sunlight, Bourneville and Letchworth, we are surprised to find the high character of the architectural treatment of these houses. Their coziness and real homelike effect contrasts very strongly with our company houses in the United States which will ever remain as one of the blots on the architectural horizon since the opportunity to combine the utilitarian with the aesthetic has been, it would appear, almost studiously avoided when it would have been possible, with a little care and study, to have approached nearer to that which we are now striving to attain, namely, cheap artistic homes for the wage-earner.

Concrete as a building material, having been used mostly in its early application to building construction and engineering work, has been left to the chaotic caprice of the uninitiated in the realms of fine arts to be developed more in a structural manner than has been good for the effect on the public at large. This has been so much so that the architectural profession has concerned itself less with the ornamental treatment of concrete than with its structural importance.

Good architectural design is based on unity, grace and proportion. It is by the adaptation of the first and last of these fundamental qualities that concrete can simulate a design which might be suitable for any other material of a plastic nature, such as stucco on brick or hollow-tile, or of even simple types of brick buildings.

Many of our present types of domestic architecture lend themselves, with very little modification, to reinforced concrete.

In the treatment of the exterior design, a style that depends upon plain wall surfaces with well proportioned and properly related openings, with the avoidance of projecting moldings, useless and meaningless ornamentations, etc., is the one which can easily be executed in concrete. It is very essential to studiously avoid molded and under-cut bands and projections. Flat bands forming string courses and panels may be judiciously used when the style of architecture demands such treatment.

Whether the roofs are to be flat or high pitched depends generally upon the climatic conditions of the locality in which the houses are to be built. We all know that the pitched roof in color lends very much to the attractiveness of a house. A low walled and high roofed house more easily produces the effect of the house being a part of the ground, or in other words, as growing out of the ground instead of simply standing upon it. This feature of making the architecture indigenous to the soil is the one which, more than any other, is so characteristic of the many beautiful industrial villages seen in England.

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* Ballinger & Perrot, Architects, Philadelphia, Pa.

It has been found to be more economical in the cost of construction to build houses in groups or rows, limiting them to rows not exceeding eight or ten, than to build single or twin houses. A great many varied effects can be obtained in the group system by the use of projecting pavilions, gables, etc.

One feature that should be given consideration in the exterior of the concrete house is its color. The unattractive mouse color of concrete cannot be made to satisfy the color instinct in man. It is for this reason that the poured concrete house, unless treated with a special aggregate or other means to enliven the color, is bound to be a failure.

Where an extensive housing program is contemplated, one method of using concrete which permits of beautiful color effects as well as artistic design is by the cement-gun. It appears that much can be gained by a study of how the gun can be brought into more universal use in house building. In connection with stucco work along the Pacific Coast it has been very successfully employed, while in the East its use in connection with engineering structures has, of late, been brought into great prominence. There is still a big field for its application in connection with houses, which the writer feels will solve many of the complications of the problem.

The use of concrete blocks for houses is restricted to certain sizes and shapes of blocks, and, although made to imitate natural material, stone, their use is unartistic from the purely architectural point of view, and for that reason have not been adopted by architects generally. Walls built of rough concrete blocks, however, can be stuccoed with hand-placed mortar. Tile inserts, properly placed, make a very effective design, especially if the colored tiles are judiciously used on piers, panels, and in string courses.

Monolithic concrete walls can best be colored by using colored aggregates, surface scrubbed, thereby removing the outer film and leaving the colored aggregates exposed to view. If it is not possible to do this, the wall can be tool-dressed with a pneumatic tool after the concrete has hardened. From the writer's experience, a light pebble forming the fine and course aggregate in the concrete, with a scrubbed or tooled finish, makes an excellent exterior for poured monolithic walls. However, as this material is not available in all localities, it is necessary to adopt a treatment of the exterior that can be obtained by materials at hand, using tile inserts as previously mentioned above.

Therefore, if we are to look for a solution of the architectural expression of the concrete house, we must get away from what has hitherto been satisfactory from a purely engineering standpoint. We must remember that a house, in order to be successful, must simulate the home spirit and, if I may use the term, have a little of that quaintness which is not found in a great many "stiff" creations parading under the name of houses. Many are merely wooden or concrete boxes.

I realize that what I have said may not be very encouraging to those whose efforts have been put forth to advance concrete house construction, but if we are to place this construction in the class of other materials which have long since been recognized and used by all peoples as meeting æsthetic demands, we will have to adopt the same fundamental principles which form the basis of all true art.

The dawn of a bright era appears to the writer to be approaching, and he trusts these few remarks will tend to point in the direction where a happy solution of this complex problem will soon be found.

INTERIOR CONSTRUCTION OF THE INDUSTRIAL HOUSE.

By Milton Dana Morrill.

In the consideration of the best interior construction for industrial houses, let us take for example these built and owned by a manufacturing or holding company to be rented by the families of the employees.

These houses must be expected to stand hard service. They will be occupied by one family after another and moving furniture is hard on the house interior. There will be some large families with several children and many with three or four lodgers. Experience shows that some of these families will keep their houses spotlessly clean while others will be indifferent as to dirt, insects, fire risks and care of the house generally.

The interior of the industrial house should fulfill the following requirements:

(1) The rooms should be light and cheerful, free from dampness, warm in winter and cool in summer.

(2) Such a house interior should be free from dust-catching woodwork.

The surfaces and finish should be as waterproof as possible so that the house may be washed out, scrubbed and thoroughly cleaned between rentals.

(3) In the construction cracks, crevices and concealed spaces should be avoided, as they are likely to harbor and offer a breeding place for vermin and insects.

(4) The construction should be fire resisting or, better, fireproof.

(5) The construction should be reasonable in first cost, permanent and capable of resisting hard usage with a minimum expense for upkeep and repair.

Let us consider three of the possible types of interior construction for industrial houses: (1) A house with masonry walls, the usual wood floor beams, board floors, lath and plaster ceilings, wood stud partitions lathed and plastered; a nonfireproof construction. (2) A house with masonry walls, reinforced-concrete floor slab covered with wood sleepers and board floor, bearing partitions of concrete or tile; fireproof. (3) A house with a concrete floor and cement finish partitions concrete or tile; fireproof.

The first type of construction is the least expensive in first cost and has the advantage of being the kind to which we are all accustomed. An interior of this kind with wooden floors, door casings, base and window trim seems the most comfortable and home-like. We would like to stop here and forget that most disastrous fires gain their headway in the open spaces and dry timbers of these wooden floors. We would also like to forget that these concealed spaces in the partitions and floors form ideal breeding places for insects and vermin. A house interior of this construction can never be completely cleaned. One untidy or careless tenant can ruin such a house. What is worse, through the neglect of proper precautions in case of tuberculosis, for instance, he might imperil the health of future tenants.

The second type of construction is an improvement over the first. The fire hazard is eliminated by the concrete floor slab. The wooden floors laid over the concrete give the house a familiar and home-like aspect. For the industrial house this type of construction has the disadvantage of being the most expensive.

The third type is a house with a concrete interior. Such a house can be made sanitary to a degree. There are no hidden spaces in the construction, therefore insects must live in the open if they live at all. On a trip of inspection to Nanticoke, Pa., the writer saw one of these concrete houses being scrubbed out from top to bottom in preparation for a new tenant. After occupancy for many years these houses will be in good condition and soap and water will make them fresh and clean again the same as it does today.

The house with a concrete interior has the further advantage of permanency and capacity to withstand hard usage. The expense of upkeep and repair therefore is reduced to a minimum and depreciation becomes almost nil except on the doors, windows, plumbing fixtures, etc. Fire insurance becomes unnecessary, as there is little or nothing about such houses to burn.

On the other hand, there are objections to the house with the usual concrete interior. During the past five years the writer has had the opportunity to inspect in all 83 houses with concrete interiors. It may be of interest to note here this list:

12 dwellings built under the Small-Harms patent in Paris, France.

8 houses built under the Roughsledge system for the Canadian Cement Co. near Montreal, Canada.

20 dwellings at Gary, Ind., for American Steel & Wire Co.

40 dwellings at Nanticoke, Pa., for the D. L. & W. R. R. Co.

1 house at Brentwood, Md.

1 house at Virginia Highlands, Va.

1 house at Union Hill, N. J., built under Mr. Simpson's plan.

The interior appearance of the majority of these dwellings is crude and unfinished, while from a structural standpoint the superiority is very marked. The question therefore arises, how may the interior appearance of the concrete house be so improved as to make it more attractive and comfortable?

Let us look into the possible changes that might be made in the finishing of the interior of a concrete house and see if some of the objections and faults cannot be overcome. We must remember that the art is new and that most of the useful things about us were not in their first stages the beautiful articles that they are today. One error that some of us have heretofore made is in trying to produce such houses at too low a cost. In the interior treatment just as in the exterior construction we can afford to go to more expense than is usual in the ordinary house, for the building is for all time. It is almost an axiom that the permanent building is the low-cost building in the long run and a concrete house with a concrete interior should with a minimum of repairs give service for a long period of years.

The cement floors have perhaps been considered as the greatest drawback in the fireproof house. There is no denying the fact that the cement floor without rugs or matting is a hard surface upon which to live; however, most of our modern hotels and office buildings as well as the modern tenements are now using cement floors and no serious complaint is made after the tenant has become accustomed to them.

If the cement floor could be given a more attractive finish, prejudice against it might be overcome to some extent. Such floors might be finished with a terrazzo surface and at a cost that is not excessive where machine grinding can be employed, or the floor may be finished with a surface of white portland cement and marble dust. A floor of this sort can be waxed and polished much the same as a wooden floor. In some of Mr. Simpson's plans he proposes using a cove base of hard wood which is let in flush with the cement. This would serve as a tacking strip for carpets if such are desired.

The treatment of the interior walls of the concrete house does not differ materially from houses of other construction. For industrial houses it is generally conceded that painting is the most sanitary and satisfactory finish for the walls. The usual wood trim or casing around the windows and doors may be used, or if desired this can be altogether omitted, the plaster returned with a slightly rounded corner, making a simple and sanitary finish.

In the industrial house, built-in fixtures and cupboards should be bracketed from the walls where possible, as this leaves the floor space clear for ease in sweeping and cleaning. Curved corners where walls meet and at the base and ceiling help in cleaning also. In time the wash-tub, bath-tub and kitchen sink for the industrial house will without doubt be molded in concrete, but at the present stage of the art enameled iron is a far better and less expensive material to use for these fixtures.

In the house with a concrete interior the stairs may be of the same material. The risers and treads can be precast and set in place. The stair railing can be made likewise. Where cost permits, decorative tile or precast panels may be set in the house walls; stenciling makes also an attractive way to apply simple decoration.

DEVELOPMENTS IN CONCRETE BARGES AND SHIPS.

By J. E. Freeman.*

Since many of the other papers to be presented at this meeting will cover questions of design and construction of concrete ships and barges in considerable detail, I have limited this present paper to a more general view of the subject with some comments on certain examples of concrete boats that may provide new information of interest.

It has generally been considered that the earliest use of what is today known as reinforced concrete was the concrete garden tubs constructed by Monier in the 1850's. But it is a fact that an earlier development in boat building by the use of cement mortar on a framework of rods or mesh was utilized by M. Lambot in 1849 in building his historic craft, the first concrete boat of which we have record. This boat was exhibited at a World's Fair in Paris in 1855 and was still seen in successful use as late as 1903.

This general method of cement plaster construction developed further in Holland when a small sloop, the "Zeemew," was constructed in 1887, to be followed in more recent days by barges of 50 or 60 tons capacity, many of the open hull type which are used on the Dutch canals for handling sand and gravel or disposing of ashes and other refuse. The "Zeemew" was reported last year in the possession of a ferro-concrete company at Amsterdam, still in good condition despite the fact that it had been in quite a number of collisions and had repeatedly been hauled up a stone embankment and frozen fast every winter. One writer says it has been used many years for duck hunting in small bays, etc.

A recent article on the Holland work in *Concrete and Constructional Engineering* (London) contains this interesting statement: "Careful examination of two concrete boats which had been in regular use for seven and eight years, respectively, showed that no growths occur if the surface of the concrete is smoothed before launching. If the surface is left very rough barnacles or algæ may adhere to it. The smoothly finished concrete also reduces the skin friction of the vessel when moving through the water. . . ."

Australian Ferryboat Landing Stage.

A concrete pontoon serving as a landing stage for ferryboats at Sydney, N. S. W., has seen severe service since 1914. During the discussion following a paper on concrete ships at a meeting of the Engineering Association of New South Wales last July, some interesting information regarding this pontoon was given by an engineer evidently connected with the Harbor Board, who said: "It measures 110 ft. in length, 60 ft. in width at one end and 70 ft. at the other, the depth being 7 ft. 9 in. It has about 3 ft. 6 in. of freeboard and displaces 783 tons. It is divided into 44 water-tight compartments, firstly, to provide against the liability to be sunk by collision and secondly, to stiffen it to withstand the continual shock of the ferryboats when berthing. The sides, bottom and deck are 5 in. thick and doubly reinforced; the bulkheads are 4 in. thick, and also doubly reinforced.

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* Portland Cement Association, Chicago.

"This being the first of its kind built here and one of the largest afloat at the time anywhere, no risks were taken, and it is perhaps stronger than future experience may warrant. It is not a ship; it has to stand more severe usage than a ship would ordinarily experience, owing to the severe shocks often given by large ferryboats, which continue day after day to bump it while berthing."

We are all more or less familiar with the Italian barges built by Carlo Gabellini among the early examples of which was the "Liguria." Many concrete pontoons from the Gabellini yards have been built for bridges across the Po and other Italian rivers, and have required little maintenance, though subjected to shocks from both vessels and ice. Doubtless this same yard had much to do with the construction of the concrete barges that are now being used by our Italian Allies for mounting naval guns of large caliber used in the defense of the Piave River line. Here is certainly good evidence of the strength of concrete under heavy shocks and vibration.

Our own country possesses today an interesting example of the Gabellini type of barge. In 1912 a group of men in Mobile built a barge about 90 ft. long, 26 ft. wide and 9 ft. deep, bringing over from Italy several engineers to supervise the construction. Doubtless owing to unfamiliarity of the owners with boat building and of the engineers with American conditions, this first craft was rather expensive and discouraged further operations, the barge remaining the sole example of their work. The barge was used for several years in handling coal, sand, etc., on the river, until driven ashore in a severe storm late in 1916, at which time a hole was punched in the side by a projecting pile or some such obstruction. Early this year the barge was repaired at small cost and turned over to a shipbuilding company at Pascagoula, Miss. The barge, though originally handling a deck load mostly, has now been carrying fuel oil in the hold—good evidence not only of the success of repairing concrete barges but of the practicability of concrete barges for the carrying of oil cargoes. This is all the more interesting in view of the present plans of the Shipping Board to construct a number of concrete oil tankers of 7500 tons capacity and the projects of several private interests for a fleet of oil barges for the Mexican oil trade. Satisfactory results in the storage of fuel oil in concrete tanks and the greater knowledge now possessed of the requirements of concrete boat building makes successful results all the more certain.

BARGE UNDER CONSTRUCTION.

Concrete barge building has now been revived in the South—the first products being the two 550-ton barges that have been built by the J. W. Thompson Co. near New Orleans, one of which is now in commission. These barges are 130 ft. by 30 ft. by 7½ ft. at the center and 8½ ft. at the rakes, drawing 2 ft. 7 in. when light and 7 ft. when loaded. Mr. Thompson, in commenting on the first barge placed in service, said: "Concrete barge 'Mila No. 1,' with 450 tons gravel, made her maiden trip yesterday. She showed no leaks at all. Handled better at loading dredge and towed steadier than our wooden barges. I vote her a real success."

Interesting progress photographs have been received of the 500-ton concrete barge being built by the Interocean Barge and Transport Co. at Seattle, and give a good idea of the general type of construction and arrangement of interior framing, reinforcement, etc. This barge is 116 ft. by 34 ft. by 10 ft. in general dimensions with a load draft of 8 ft. The hull is divided into water-tight compartments by one longitudinal and four cross bulkheads. These are 4 in. thick, while the sides, bottom and deck are 3½ in. The barge has a weight of 350 tons, which gives a ratio of dead weight to displacement of about 60 per cent. Concreting is now completed after continuous operation from start to finish with the exception of a break at the deck line where a joint was made. It was reported that after over one hundred hours continuous placing the concrete gang stopped for five minutes to cheer the good ship "Faith" as she steamed up to Seattle on her maiden voyage—certainly a sufficient excuse for a few moments delay.

In the East the 700-ton barge built by the Louis L. Brown Co. has been launched and other barges are now being built for the Navy Department, while there is every prospect of a large number of barges of this kind being built for service on the Erie Canal and other inland waterways. Concrete barges have proved their usefulness. The 200-ton "Pioneer" built in 1910 for use on the Welland Canal is still serving just as efficiently as when new, and has been subjected to severe tests. The barge has been loaded many times with carloads of rubble stone dumped from a height of 12 to 15 ft. upon the concrete deck, the full load starting from one end, which procedure would doubtless make an old-time wooden-scow captain shiver. Answering a question recently in regard to the effect of loading stone thus, J. L. Weller, the designer and builder, has said: "The deck of the 'Pioneer' is of 2½ in. concrete and has never been protected or covered in any way while stone was being dropped upon it. In probably half a dozen cases where a large stone dropped on its corner a dent was made on the surface, but this did not amount to anything and apparently has been patched up, as they are not noticeable, and the deck is now 99 per cent, at least, in as good condition as when built."

The need is great for river barges to handle coal. About 50 per cent of the wood coal barges used on the Ohio River for this purpose were lost in the ice jams last spring. We have given some study to this problem and have worked out a tentative design for a 500-ton concrete barge that may be of interest to those studying the same subject. The displacement, loaded, is 680 tons, giving a ratio of dead weight to displacement of 70 per cent.

The barge has the same length and breadth as the wooden barge, *i. e.*, 135 ft. by 26 ft., and a slightly greater depth and is intended to be used in the fleets or separately as required. Results of experiments made by the Government and published in a bulletin, "Experimental Towboats," indicated that for the draft and conditions under which this barge would be used the circular type of rake offered the least resistance in towing.

Brackets and cross beams 9 ft. apart support the shell and top beams, but the brackets project only 2 ft. inside, which should interfere little with loading or unloading. There are also longitudinal beams in the bottom, which is a double bottom for extra safety against sinking if the bottom is damaged and gives extra stiffness also. The shell is $2\frac{3}{4}$ in. on the bottom, decreasing to 2 in. at the top. The draft is 22 in. light and 6 ft. 7 in. loaded (concrete 155 lb.). With the use of lighter weight concrete this could be reduced.

The floor might be cast with the rest of the barge or made of precast sections placed later, thus simplifying forms, a wedge-shaped space left between the edges of adjacent slabs over the beams containing strap-iron anchors which, when encased in the concrete filling the wedges, hold the floor slabs securely. The wedges could be cut out when necessary to replace individual sections, the edges of the slabs being coated with tar.

A barge somewhat similar to this type has been used on English canals for handling ashes, etc.

FOREIGN PROGRESS GREAT.

England and France are making great strides in the construction of barges and lighters, many of them for sea-going service and others for the various canals and waterways. A good example of what a standard design and standardized forms can accomplish in the way of speed of construction is found in the yard of a French company located on the bank of the Seine. Here as many as 10 lighters about 125 ft. long are under construction at a time and production is said to average one vessel a week. The forms have been carefully designed not only to give the craft graceful lines but to facilitate assembling and taking down, and of course are used repeatedly. It is stated that about ten days time is required to set the forms and place the reinforcement and two days to place the concrete, working continuously. After forms are removed the surface is smoothed by rubbing. When the craft is to be launched a large crane picks it up bodily and places it in the water. This crane also serves a useful purpose in handling forms, etc.

T. J. Gueritte, the French engineer whose paper on concrete boat-building was recently published in various technical journals here, says in it: "The author has seen in what was four months before a bare field, two 1000-ton barges ready for launching and several others following closely." He further makes the interesting point that with concrete, alterations in the design in course of erection can be readily effected in building both barges and ships of concrete, which undoubtedly has its advantages in some cases.

James Pollock and Sons Co. of London, have adopted the use of straight lines wherever possible for the small coasting vessels now building, to simplify formwork, but the J. and B. Stewart Co. of London and Belfast, which has completed recently a sea-going barge, followed the usual ship lines. The vessel was apparently constructed in a drydock or slip and launched by filling the dock, which is somewhat the same procedure as has been proposed by some in our own country.

The 125-ft. concrete ship built by the Atlas Construction Co. of Montreal and launched last November, is practically ready for service. This vessel is interesting because of the use of structural steel ribs which were encased in concrete at the time the concrete shell was cast.

The Norwegian yards, which were among the first to build concrete ships for ocean service, are continuing their activities. The Fougner yard at Moss completed in March, a 600-ton motor ship similar to the "Namsenfjord," which is well known to all present, and is following this with 1000 and 1500-ton vessels, while the Fougner American yard is at work on concrete barges and several 3500-ton ships. This company built the first floating drydock of reinforced concrete last year for a Christiania firm of yacht builders, the dock accommodating a vessel 75 ft. by 25 ft. and having a lifting capacity of 100 tons. The application of concrete to a floating structure of this kind is particularly interesting in view of the need for many floating drydocks in connection with our rapidly expanding merchant marine.

The Porsgrund Company of Porsgrund, Norway, has also another lighter nearly complete similar to the 200-ton vessel launched last July, which is unique in that it was constructed and launched bottom up. The vessel was righted in the water by an ingenious arrangement of inner compartments whereby water was admitted to certain compartments while others remained filled with air, thus unbalancing the craft so that it slowly turned over until it floated in correct position.

THE AMERICAN-BUILT "FAITH."

But it has remained for America to take the lead by the construction of the 5000-ton concrete cargo steamship "Faith" on the Pacific Coast and demonstrate that concrete was practicable for ocean-going vessels of large tonnage. It is tonnage of this class that we need now—this year—which by reinforced-concrete construction can be supplied quickly and without interference to the construction of wooden or steel vessels.

Our lead should be maintained and a comprehensive program of concrete ship construction vigorously prosecuted. The attitude of our English Allies toward this matter is strikingly emphasized in a remark of one of the Admiralty officials concerning concrete ships, that, "Every encouragement is being given by the Admiralty; newly-formed yards have been laid down specially, while existing shipyards have been encouraged to form branch establishments for the purpose."

When the construction of this vessel was begun in September, 1917, many were beginning to realize the magnitude of the shipping problem, but few had given thought to the possibilities of concrete construction in solving that problem. The construction has been quite generally described in print and will be shown in the motion pictures, so I shall not dwell on that here.

The shipping problem had become more fully understood by the time the vessel was launched on March 14th and christened "Faith" and the success of this launching attached greater interest and importance to the concrete ship.

Early in May the "Faith" was completely fitted out ready for her trial trip—a record for such work on the Pacific Coast—and during the trials developed more than the designed speed of 10 knots which it was estimated the 1750 h.p. engines would give.

The latter part of May the "Faith" took on her first cargo, salt and copper, and a few days later sailed for Vancouver *via* Seattle and Tacoma.

The maiden voyage of the "Faith" was as strenuous and severe a test as could be asked for in a concrete ship and she stood up under the gale, which at times put the decks awash, in splendid fashion, reaching Seattle—her first port of call—six days after leaving San Francisco, in excellent shape, having taken in no water through the bottom.

Faith in concrete has again been justified. The application of a tried material to a known problem has again been accomplished. Concrete ships must be built in large numbers and with all possible speed to help answer the call for ships, more ships and still more ships, and win the war.

THE YARD FOR BUILDING CONCRETE SHIPS AT WILMINGTON, N. C.

By Archibald G. Monks.*

There is little experience yet in design of concrete ships and less in design of plants as compared with the well-established building of steel or wooden ships. Changes and improvements may confidently be expected. At this date the reproduction in concrete of the framing and the form of steel ships has been wisely adopted, and has been followed by the British engineers and is advised by the Joint Committee of the American Concrete Institute and the Portland Cement Association and conservative naval architects and engineers.

Controversy over the merits and demerits of design of ships, concrete for use in sea water, comparative costs of maintenance, initial costs of net cargo capacities with per ton costs of transportation, ultimate length of safe service and the final place the ships may occupy in the economics of ocean transportation are not good questions for forcing upon the now heavily overloaded public mind. They are still open questions whose existence may well stimulate the keenest rivalry for their solution by those who are chosen for that work, and, as well, those who are technically capable and have not yet been delegated to some phase of the larger question.

The Liberty Shipbuilding Co. of Boston has one of the five agency contracts for concrete ships. These will be built at the yards at Wilmington, N. C. I desire to present briefly a description of this yard.

Yard Buildings of Light Construction.

The fact that concrete shipbuilding is still in the pioneer stage, has determined the character of construction of the yard; this will be temporary with light wooden construction. Rapid changes in yard design and equipment and larger salvage of materials of yard and equipment will be possible, more rapid prosecution of the first shipbuilding itself can take place, and less initial cost will be incurred. An example of possible change of yard design is the use of steel forms (as the art of concrete shipbuilding advances). The design of the yard will also be governed by local conditions, the site, the market for building materials, the number and size of vessels to be built and the climate

Discussion of this yard divides itself naturally into groups: the site, water basins and shipways; the concrete department, erecting machinery and the general storehouse; the traffic service of standard and narrow-gage railways, paved truck roads and rolling stock; the timber department of storage, open and enclosed and wood-working buildings and machines; steel-storage and steel-working buildings; the general yard service of water, fire protection, sewerage and electricity and the provisions for the main office, the yard employees and the troops to be stationed at the yard.

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*Monks & Johnson, Consulting Engineers, Boston, Mass.

The site is in the City of Wilmington, N. C., on the Cape Fear River about 30 miles from the sea. A deep channel has recently been dredged by the site to a point above the city. The top soil is marsh mud some 15 to 20 ft. deep overlying sand and gravel base. The channel dredging furnished material for hydraulic sand fill over considerable portion of the thirty-five-acre site. Additional dredging between channel and site and from the launching basins will provide further fill. The site has a river frontage of 3500 ft., about 1400 ft. of which is now being developed to a depth of 1100 ft. Within this space provision is being made for four ways in the form of three launching basins with two piers, having launching ways on both sides, parallel to the basins. The basins or docks are 460 ft. long and 160 ft. wide and are dredged to a depth of 20 ft. at low water. The piers are 350 ft. wide. Wooden piles with heavy timber capping, support the ships during construction and launching. A heavy bulkhead at the pier edge retains the sand fill which forms the working deck. The pier slopes from high water level at opposite edges, at the rate of 1⅛ in. per ft. until it reaches the level of the central portion 5 ft. above high water. The tankers are to be 420 ft. long, 53 ft. wide. They will be launched sidewise

Storage of concrete materials with capacity for one ship will be on the piers between the building slips so as to serve a ship either side. The cement will be stored in bags in a wooden shed, and the sand and stone will be piled in the open.

Two single cubic yard concrete mixers, mounted on wheels, will be moved from ship to ship. When in operation, they will be placed between the materials and the ship. A spare mixer will be provided

DERRCKS HANDLE MATERIAL

Two shipyard derricks located at the shipway will handle the form, reinforcing steel, steel and iron castings, and concrete. These derricks will be of three-ton capacity at 75 ft. radius with towers 50 ft. high. Both will be in operation on one ship during the pouring of the concrete. At other times they will be used among the ships under construction as required. These derricks travel on rails 24 ft. apart and have a wheel base of 24 ft. They can be moved from ship to ship by means of a transfer car running along the head of the docks. This car will be made strong enough to enable the derricks to be used while on the transfer car in unloading materials from lighters in the docks.

A general storehouse will be provided at the head of the basin between two piers, thus serving four ships. This will be an enclosed wooden building to house general supplies, rigging, tools, and spare parts for the yard machinery.

The main line of the railroad enters the yard at the rear at one end and runs parallel with the river throughout the extent of the present development and can be extended if the yard is enlarged. From the main line spur tracks are run to the lumber storage and to the piers. One spur track runs through the steel storage and bending space, another runs in front of the general storehouse and along the form storage; other spurs serve the mold loft and carpenter shop. Thus every process in the yard is thoroughly served with standard-gage railroad, except that the railroad used to take lumber from the storage piles to the saw mill and the carpenter shop is narrow-gage. Paved roads parallel the railroad everywhere and thus when cars or railroad may not be available, automobile trucks will be used instead.

The yard will be provided with a small number of standard-gage railroad cars and a locomotive. A larger number of narrow-gage cars will be provided for handling lumber. The number of automobile trucks which will be required to supplement railroad transportation has not yet been determined.

Rough lumber storage is beside the yard's service railroad, and is in the open except as experience may call for roofed-over portions; adjoining is the saw mill for cutting to required sizes and for dressing; then lumber is taken to finished lumber storage with finished lumber purchased directly.

The mold loft is one story, 90 ft. by 260 ft.; the central bay 60 by 260 ft. clear of posts; flooring of white pine carefully leveled constitutes a full size drawing board on which the ship's lines are laid out and from these the molds or templates are made

DETAILS OF BUILDINGS.

The carpenter shop is a one-story wooden building 60 x 200 ft. with an ell 60 x 80 ft. near one end in which wood-working machinery is located. At the other end of the shop is a high portion in which bow and stern forms are built to full height. The machinery ell has enclosing side walls. This shop is located near the mold loft from which come the molds according to which the forms are built

The oil-process building will house oil-dipping tanks and drain racks. The lumber for the facing of the molds will be the best seasoned stock obtainable. It will be shipped to us in closed cars, then dipped in linseed oil, then, as assembled, all joints are to be painted. The finished sections of flat forms will be dipped and the surface that has contact with the concrete will receive a final coat of paint.

Form-storage buildings are to be merely shelter roofs and will be located close by the building ways. They will have enough space for the storage of forms for four ships. The flat pieces and panels will be piled up but the large built-up forms for the curved surfaces cannot be piled.

Reinforcing steel will be taken off the cars and placed in storage racks and will be sorted to size. Space will be provided for four thousand tons. About twelve hundred tons are required for each ship.

Cutting shears and bending tables will be under a shelter roof; cut and bent steel will be taken on standard-gage cars and stored in the open alongside the railroad tracks (at convenient places).

Water service, for manufacturing, drinking and sanitary purposes will be provided from the city supply. The city will bring this to the yard in a 6-in. main and the shipbuilding company will provide its own distributing system

Water for fire protection will be pumped from the river probably into an elevated tank to supply the automatic sprinklers and yard hydrants. It is expected that all buildings will be protected by automatic sprinklers, well placed hydrants and hose houses.

A city sewer now runs through the site and empties into the river. All sewerage and drainage of the yard will connect with this sewer or will empty directy into the river

Electricity both for power and for lighting will be purchased from the local public service company who will completely install it.

An additional "breakdown" power station having capacity enough to light the yard (in case of a breakdown during the concreting of a ship) will be provided. The concrete is to be poured continuously from start to finish to avoid construction joints and laitance. It is expected that the pouring of one ship will take three or four days' continuous work. The lighting service covers the entire yard, the buildings and the protective fence about the yard.

Electric power will be used for the wood-working machines and for the steel-bending machines. The mixers and derricks have their own steam power.

The gate house, employment office and first-aid rooms, are of more or less standard arrangement. The gate house designed to pass two thousand men.

The restaurant is to be two-story, wooden, fully supplied with screens, with kitchen on first floor and seats for 250 men—provision has been made to double the capacity, if desired. Service will be cafeteria, each customer taking a tray, passing the service counter, then the cashier's deak, before eating. The second floor has a lunch room for the office force and a conference lunch room for department heads.

For the troops to be stationed at the yard, barracks with the requisite number of additional buildings, officers' quarters, cook-house, mess-houses, etc., will be built. The presence of uniformed soldiers at a shipbuilding yard has been found to be important and beneficial, not only for the protection of the yard from marauders, but as a constant reminder to the men of the cause for which the shipbuilding work is being carried on.

The office building is two-story, wooden, for the executive and clerical force of the Liberty Shipbuilding Co., its engineers and superintendent and the government engineer and auditor

SIDEWISE LAUNCHING USED.

It was noted earlier that the ships are to be launched sidewise; one of the interesting features discussed between the Emergency Fleet Corporation and ourselves is the construction of the ways for side-on launching. It has been determined that this is preferable to end-on for several reasons, among them being that side-on can be used on narrow channels, and that a ship may be built on a level keel with greater ease in assembling forms, placing of steel, pouring of concrete and generally in the construction

In end-on launching the stern of the ship entering the water first floats before the bow clears the ways. The ship is thus suspended by the bow and stern, and severe stresses are induced. The stresses of side-on launching are very much less severe. Again, a steel ship hull has attained its total strength at launching, but the concrete hull needs a longer period to reach its maximum strength. Side-on launching has been used considerably on the Great Lakes and in other places, and is used on the Clyde for steel vessels due to the narrow channel

The disadvantages of side-on launching which some regard as sufficient reason for preferring end-on, are that it is more difficult to reach the outboard side of the ship and to work there, and enclosing structures to provide protection from the weather, and supports for overhead traveling cranes, is a great deal more expensive, and the whole of one side has to be built removable while in the case of end-on launching the doorway is much smaller

Roofing over the ship during construction is necessary in the South on account of frequent showers, often followed by strong sunlight. This is particularly important during the process of concreting.

DESIGN AND CONSTRUCTION OF A CONCRETE BARGE.

By L. L. Livingston.*

It is a well-known fact that one of the first uses of reinforced concrete was in boat construction. However, the literature on the subject is very limited, and while a number of attempts have been made to build boats out of concrete, the necessity for using every available material for shipbuilding purposes has never been so great as at the present time.

The first concrete boat built by the Louis L. Brown Co. was a deck barge 112 ft. long, 22 ft. wide, 10 ft. deep and has 18 in. shear and 2 in. camber in the deck. Fig. 1 shows a longitudinal and a cross section of the boat as originally designed. Slight modifications in certain details, and in the arrangement of the bulkheads were made during construction. The boat is divided into six compartments instead of the four shown by one longitudinal and two transverse bulkheads. The bottom and deck beams are carried by longitudinal reinforced-concrete trusses. The panel length of these trusses was 7 ft. center to center, and the depth equal to the depth of the boat. They are spaced 5 ft. 4 in. apart across the width of the boat and are reinforced to take care of any hogging or sagging strains due to the following conditions:

1. The central third of the boat loaded with the end thirds light.
2. The end thirds loaded and the center third light.
3. One-third the load on one end of the boat.
4. The ends of the boat supported on the crest of a wave equal in height to $\frac{1}{20}$ its length.
5. The center of the boat supported on the crest of a wave equal in height to $\frac{1}{20}$ its length.
6. The boat light supported in the extreme ends with no buoyancy.

The bottom, sides and deck slabs are $2\frac{3}{4}$ in. thick supported by beams 3 ft. 6 in. center to center. Fig. 2 shows a plan of the deck with the location of the hatches, bits, fittings, etc. The cabin, deck, bulkheads and fenders are built of wood. The stress used in the design are:

Reinforcing steel—16,000 lbs. per sq. in.
Extreme fiber stress in concrete—800 lb. per sq. in.
Allowable shearing stress in plain concrete—50 lb. per sq. in.
Allowable shearing stress in beams reinforced with bent-up rods and stirrups—125 lb. per sq. in.

The boat was built for side launching. The building ways were spaced 14 ft. center to center. The forms were built in panels and bolted together. The bottom panels which were carried by the building ways were supported at the ends by wedges, which permitted them to be removed before removing the building ways.

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* Louis L. Brown Co., New York City.

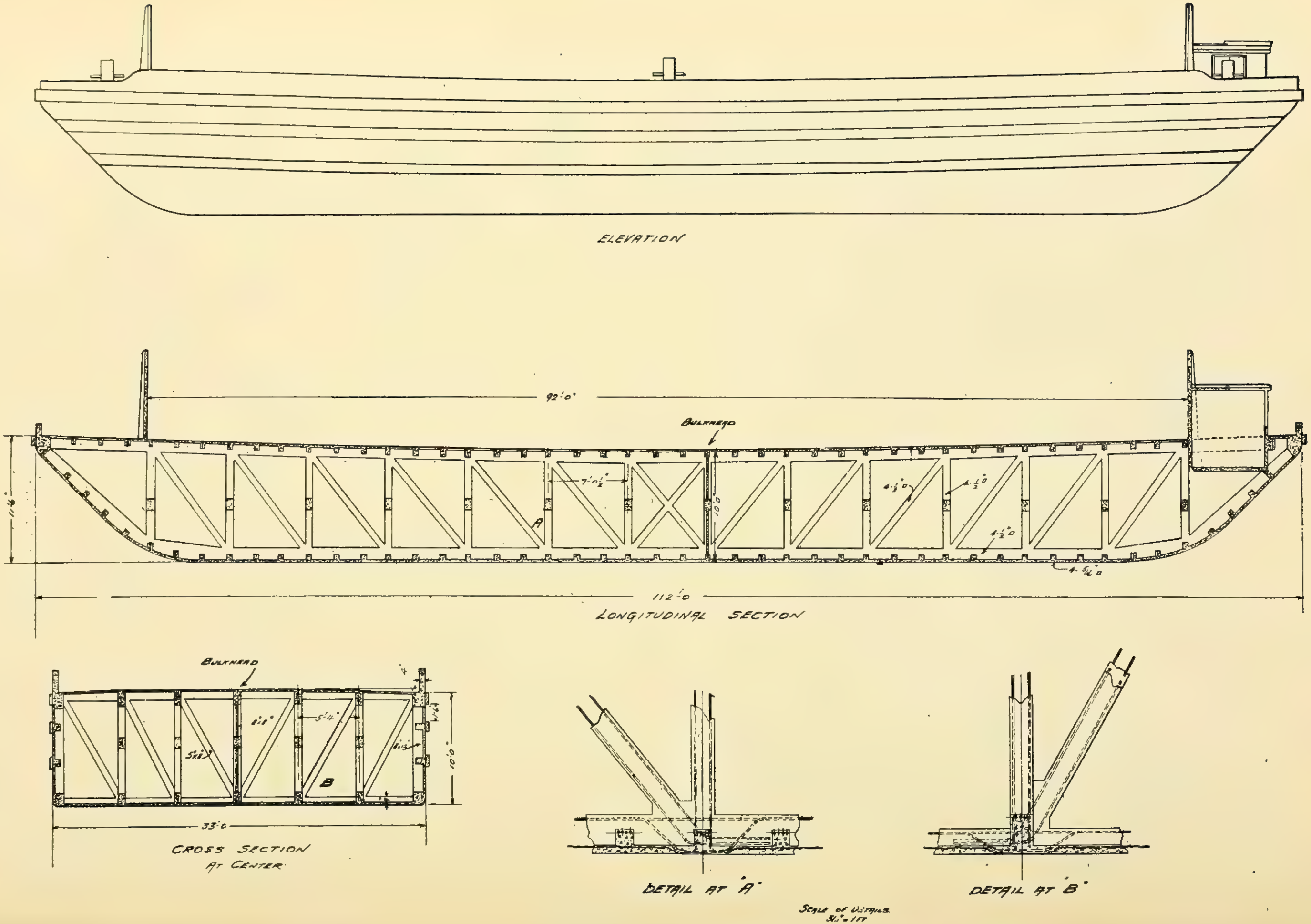

FIG. 1.—LONGITUDINAL AND CROSS-SECTIONS OF 725-TON CONCRETE DECK BARGE.

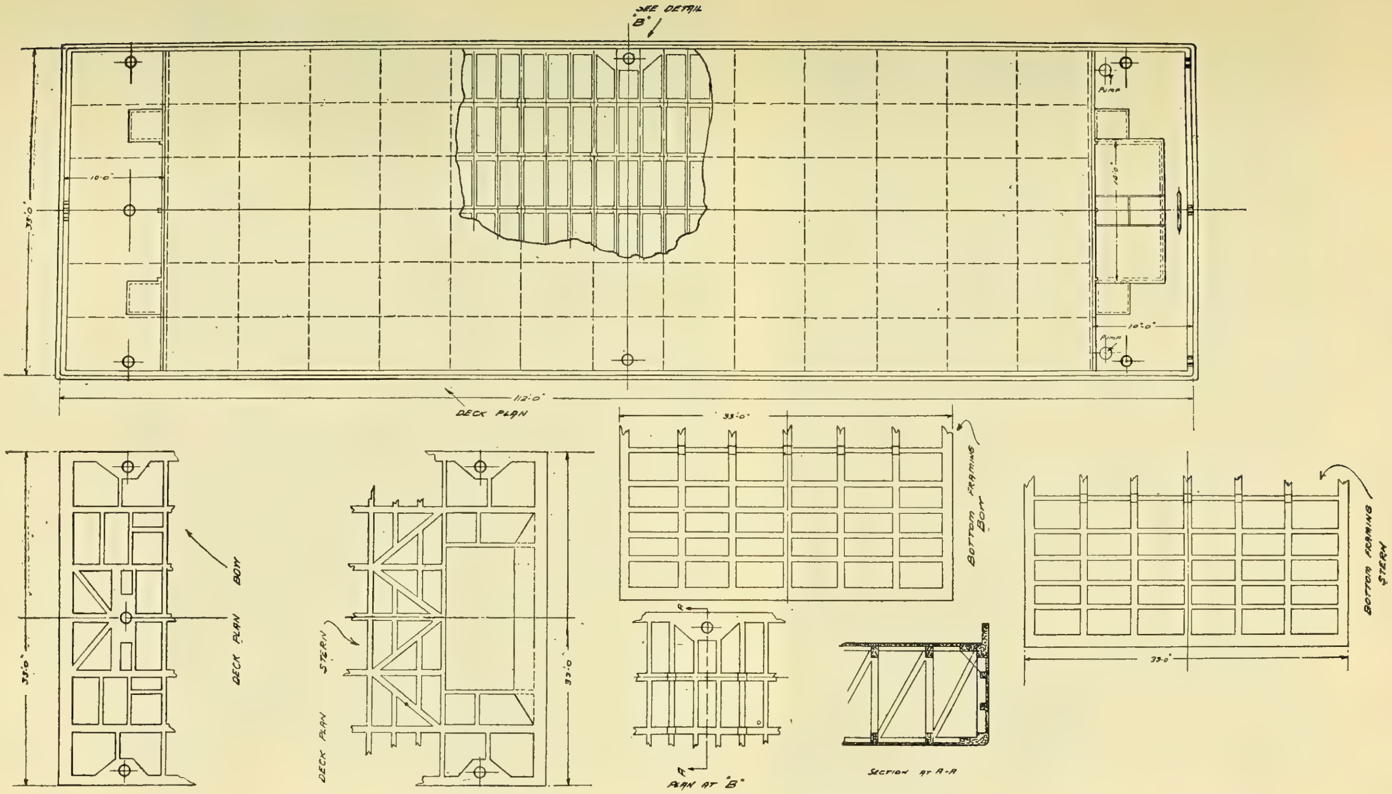

FIG. 2.—DECK PLAN AND DETAILS OF CONCRETE BARGE.

As the cement gun was used in concreting the sides and bottom only one form was necessary. This was placed on the outside of the boat. The materials used were mixed dry in a batch mixer and conveyed to the guns, located so as to cover the entire boat. Some of the views show the outer forms.

All materials were tested and favorably reported for use for concrete in sea water by the Pittsburgh Testing Laboratory.

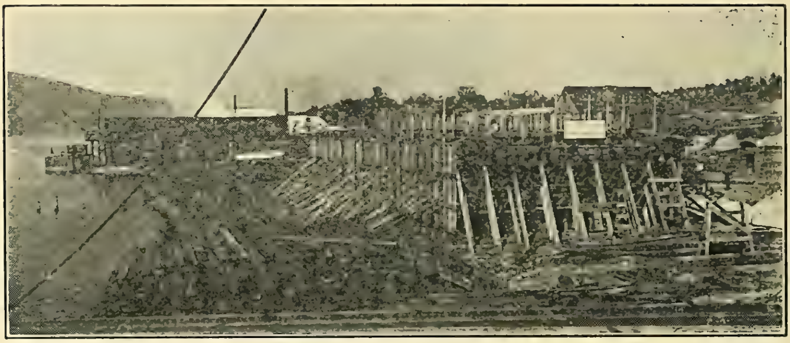

FIG. 3.—CONCRETE BARGE UNDER CONSTRUCTION.

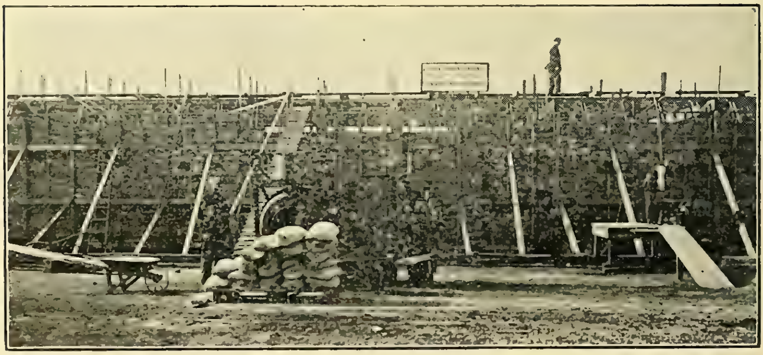

FIG. 4.—CONCRETE MIXER AT WORK FOR CONCRETE BARGE.

The proportions used in the mix varied. In the bottom it was one part by volume of cement, $1\frac{1}{2}$ parts by volume of sand and two parts by volume of limestone screenings. In the sides the mix was one part by volume of cement to $1\frac{1}{2}$ parts by volume of sand. In the interior beams, trusses and deck it was one part by volume of cement, 2 parts by volume of sand and two parts by volume of limestone screenings. The limestone screenings were screened through a $\frac{1}{4}$-in. screen and were well graded in size. Giant portland cement was used throughout in the construction of this boat.

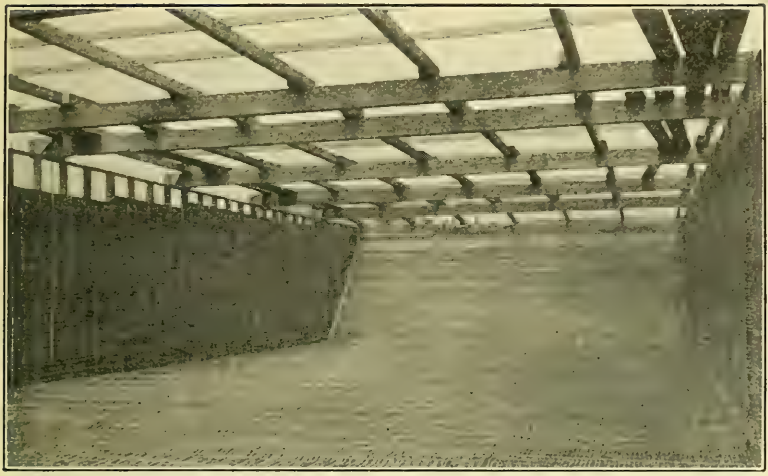

FIG. 5.—INTERIOR OF BARGE WITH MESH REINFORCEMENT IN PLACE.

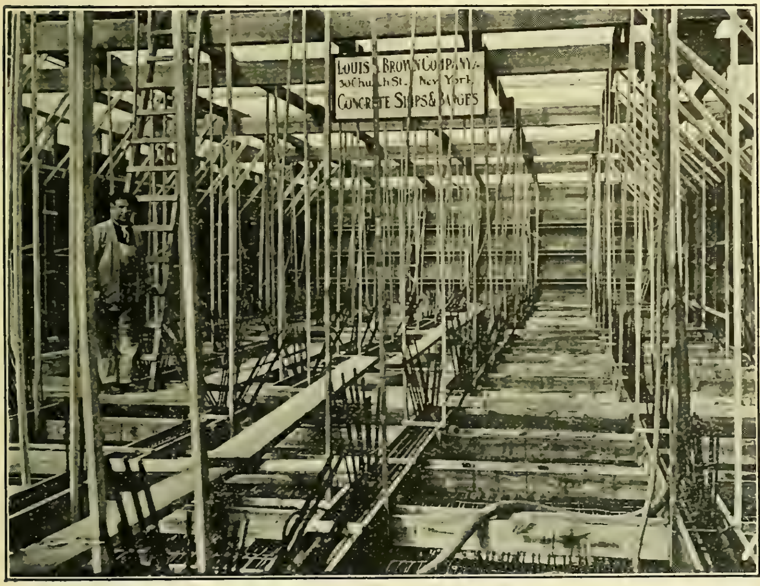

FIG. 6.—REINFORCEMENT OF INSIDE FRAMING IN PLACE.

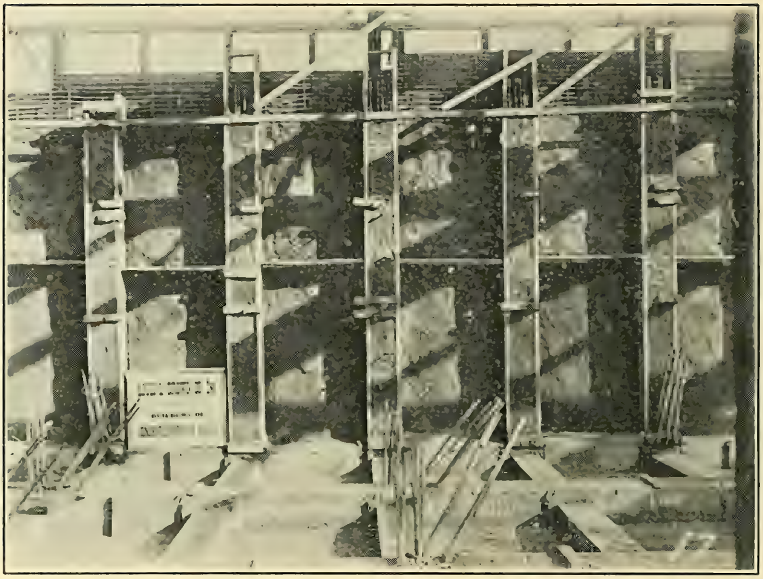

FIG. 7.—SIDE FRAMES CONCRETED WITH CEMENT GUN.

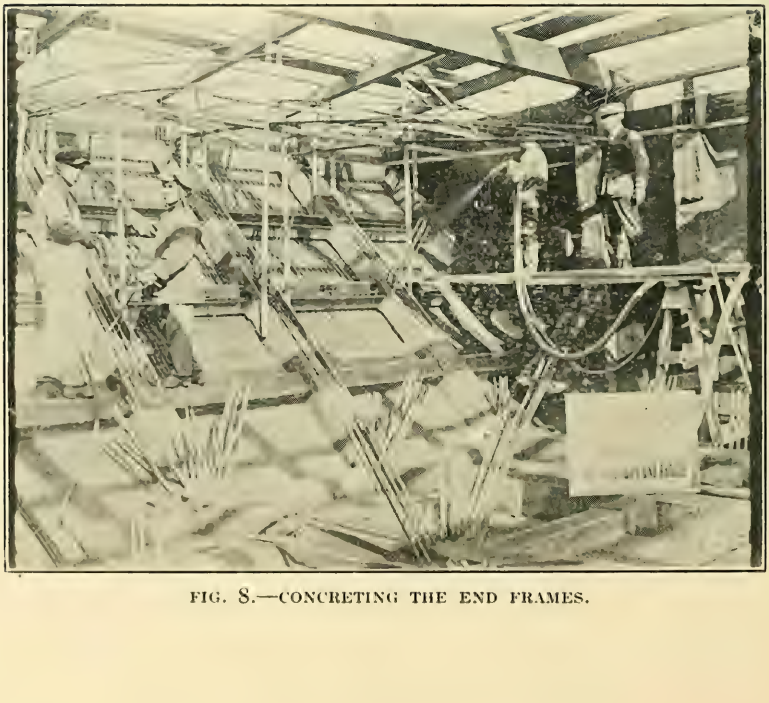

FIG. 8.—CONCRETING THE END FRAMES.

The reinforcement used in all slabs and bulkheads was Clinton Electric weld galvanized wire mesh No. 3 and 8 wire spaced 2 x 16 in. One view shows the wire mesh in the hull of the boat. All reinforcing rods used in the beams were Havemeyer bars. This reinforcement together with the forms for the bottom beams and the ribs in the side were supported from the top by beams spanning the width of the boat as shown by Fig. 5.

The concrete was applied in the outer hull in three operations. First, the bottom slab and beams were concreted. Second, the side slab and beams for

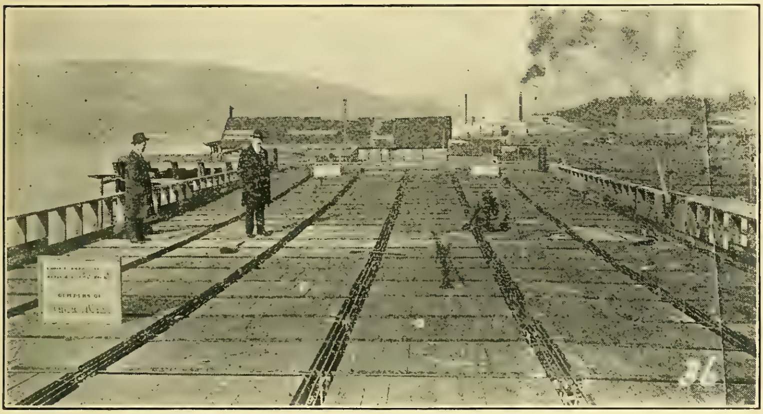

FIG. 9.—DECK FORMS AND REINFORCEMENT IN PLACE.

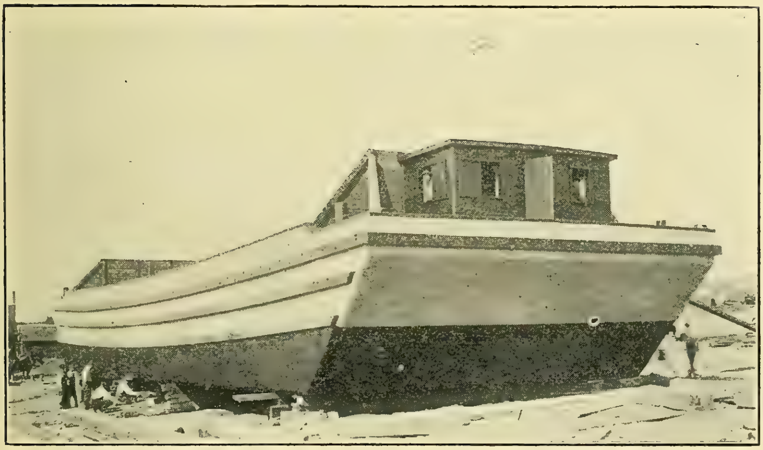

FIG. 10.—THE COMPLETED 725-TON CONCRETE BARGE.

approximately one-half the height of the sides and the third operation completed the sides.

Fig. 7 shows the bottom and sides concreted. Fig. 8 shows the two operators concreting the end rake.

It was necessary to concrete the sides in two operations as it was found in attempting to concrete a vertical slab of this thickness with the cement gun for a greater height than about 5 ft. that there was a decided tendency for the concrete to drag our pull down causing it to crack before it had set.

The remainder of the boat was poured in two operations. Fig. 9 shows the deck forms before placing the slab reinforcement.

The bitts were made of 10-in. pipe with holes through the bottom and sides for reinforcing rods, which spread out on the deck.

Three sets of 6 x 8-in. fenders were attached to the sides of the boat by bolting through the side ribs. Holes were provided in the concrete for the bolts by wooden pegs placed in the forms before concreting. After the forms were removed the pegs were bored out of the concrete.

Marine glue made by the New-Pro Chemical Company was heated and poured into the bolt holes and the bolt for attaching the fenders was inserted while the glue was hot; the glue filling any space between the bolt and the concrete.

Fig. 10 shows the boat completed and ready for launching

It was intended to launch this boat in the fall of 1917, but upon removing the outside and bottom forms, it was found that the product obtained by the use of the cement gun was not sufficiently uniform to insure a water-tight hull. It was necessary to go over the boat very thoroughly and remove the sand pockets or defective concrete and replace them with new concrete.

While the writer believes that it is possible to build a water-tight hull with the cement gun with less sand pockets than was encountered in the construction of this boat, he does not believe, even under the most favorable conditions as to the design of the boat and in the operation of the gun, that sand pockets can be entirely eliminated. After all the defective concrete was replaced, the hull was painted with a neat cement grout, which was thoroughly rubbed into the pores of the concrete.

Four launchways were used in launching, spaced so as to divide the load as equally as possible. The declivity of the ways was $\frac{3}{4}$ in. per ft., and the pressure was approximately $2\frac{1}{2}$ tons per sq. ft. of way. The launching was very successful, and the boat draws 3 ft. 8 in. of water when light, and shows no leakage.

After having made a thorough study in the design and in the methods of construction of concrete barges, the writer believes that there are no difficulties that cannot be overcome, and that eventually reinforced concrete will prove to be the desirable material for the construction of barges and other floating craft, both from the point of view of first cost, and for serviceability.

PROBLEMS ARISING IN THE DESIGN AND CONSTRUCTION OF REINFORCED CONCRETE SHIPS.

By R. J. Wig* and S. C. Hollister.†

It is presumptuous to attempt to outline in any complete manner the problems which are to be met in the design and construction of concrete ships before we have had the actual experience of completing the construction of some such ships.

This paper is prepared not with the intention of telling you just how reinforced-concrete ships should be built, but rather to acquaint you with some of the many problems which must be given consideration and which we must look forward to solving if we are to make more than an emergency success of the concrete ship.

Reinforced-concrete floating craft such as scows, barges, and the like, were first built many years ago. The first reinforced-concrete cargo motor ship was not launched until August, 1917, less than one year ago. This ship was built by N. K. Fougner at Christiania, Norway. Since this time many small vessels of 1000 tons dead-weight or less have been built in Europe.

The pioneer of the large reinforced-concrete ship is the San Francisco Shipbuilding Co., which launched the 5000-ton deadweight ship "Faith" in March, 1917. This ship is the first and only cargo motor ship built in the United States.

Today there is scarcely an engineering office in which a design of a concrete ship has not been started, and it is the common topic for discussion among engineers. Most engineers find, however, that the design of a ship is a very complicated and intricate problem, involving, as it does, all branches of engineering experience.

A ship is in reality a floating hotel, power plant and warehouse combined, and one no sooner starts its design than he is impressed with the tremendous amount of detailing required and the hundreds of fittings and machinery parts which must be given consideration and attention throughout both the design and construction.

Structural Design.

A few computations in ship design make it evident that it would not be economical to design a concrete vessel, using the concrete unit stresses commonly employed in commercial building practice. This has the immediate effect of demanding for ship construction the highest available quality of concrete. This demand is actuated by the requirement of maximum ship strength per unit of weight of material. A considerable advantage may be effected by the development of a light aggregate of high strength. Some work has at present been accomplished in this direction and will be referred to more in detail later. After once deciding upon the ultimate strength of the concrete, it is then necessary to decide upon the working unit stresses and the factors of safety to be allowed.

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The department requires a concrete having an ultimate strength of 4000 lb. per sq. in. in 28 days. With this as a basis, the adopted unit stress in outer fiber of 1500 lb. per sq. in. is 37½ per cent of the ultimate. A bond of 200 lb. per sq. in. is allowed on reinforcement deformed to provide a positive mechanical bond. The tensile unit stress in steel is 12,000 lb. per sq. in. in places where the formation of cracks would lead to seepage, and is 16,000 and 20,000 lb. per sq. in. in places not exposed to the water, the highest value being used in the intermediate transverse bulkheads.

NAVAL ARCHITECTURAL DESIGN.

The first consideration that arises in the development of design of a concrete vessel is its form or lines. Many have advanced the belief that concrete ships should be formed with extremely simple lines: with square knuckles and bilges, and with surfaces curved in only one direction. The reason they offer for this is the resulting simplicity in form work. Others believe that this same argument would apply to the bending of plates in a steel vessel, and that the concrete vessel is not unique in presenting this problem. Furthermore, they claim that there is no reason to believe that after a type of form is decided upon, the types should be more difficult to build on long easy curves than with flat surfaces. The steamship "Faith," which is the first large concrete vessel, has these extremely simplified lines. The vessels at present under construction by the Government have faired lines with surfaces curved in more than one direction. The behavior of these two types of forms will probably give considerable information for future design. It is well known to naval architects that simplified lines are not a hindrance to speed, but rather, if properly designed, may give very favorable results.

The proportions of the concrete vessel are somewhat different than of the steel vessel. Where the side walls are heavy, the effect is to slow down the period of roll because of the inertia of the sides about a longitudinal axis. This in turn would seem to permit the use of a greater metacentric height than is common in a steel vessel designed for the same service.

The weight of the concrete vessel is in excess of the weight of a steel vessel of equal carrying capacity and necessitates a careful consideration of the arrangement to gain a maximum strength with a minimum volume of material. It may be that some future designs will develop to high degree a system of framework and general arrangement which will more nearly suit the nature of the material than the common type of steel arrangements when made up in concrete. Concrete is capable of higher compressive stresses than tensile stresses and this reason would seem to make desirable an arrangement which employs to the greater extent the compressive strength of the concrete.

Computations for stability of concrete vessels do not differ from those made for steel vessels. Developments up to the present time have not introduced any new considerations in the determination of the vessel's stable quality.

EFFECT OF VIBRATION ON CONCRETE SHIPS.

There are a number of peculiar actions in the life of a ship which have caused considerable discussion among ship builders and ship operators in general. One of these is the effect upon concrete of the vibration of the engines. It has been thought by many that the vibration would slowly disintegrate the concrete. Those familiar with concrete construction on land may well realize that this is not in general true and that there are many fine examples of concrete structures subject to a considerable vibration without any apparent deterioration. It has been found, from the concrete vessels at present afloat on both sides of the Atlantic, that the concrete ship exhibits less vibration as a structure than wood or steel and in some instances the vessels are practically free from vibration whatsoever.

It is planned to carry out extensive investigations on the completed concrete ship to determine as nearly as possible the points of greater stress. Pressure-recording instruments will be placed at different points on the ship to record the hydrostatic pressure exerted upon the vessel. This work is being carried on under the direction of Prof. F. R. McMillan, formerly of the University of Minnesota. Recording strain gages will likewise be placed at critical points to measure the deformations in the material. Some of these will be placed upon the members of the transverse frames, while others will be placed on the side wall to measure the deformation in the side shell reinforcement. Measurements will also be taken upon the longitudinal steel in the deck and bottom. Attempts will be made to measure every possible torsional strain occurring in the vessel. There are several places in the vessel where there is a congestion of stress. One of these points is at the corner of hatch openings. Another point is in the gunwale at either end of the shell, forming the side of the bridge erection.

One problem of seaworthiness of the concrete vessel is the resistance it may offer to pounding and panting. The first term is one applied to the action of a boat whose forward or after end has been raised out of the water and then smashes down onto the surface with a tremendous impact. The second term applies more generally to the flexible action of a steel vessel and describes the diaphragm action of the plating as the external hydrostatic pressure alternately increases and decreases. Just what the resistance of the concrete shell will be to such sea conditions remains to be seen. It is only logical to suppose, however, that the concrete shell will not be so apparently flexible as the steel shell and that, therefore, panting will not be apparent. Similarly it would seem that a considerable reliability may be placed upon the strength of the concrete frame under impact in the light of our present knowledge of structural action in general.

WHAT TYPE OF SHIP TO USE.

The type of vessel more suitable to concrete would seem to be a vessel which has the greatest possible continuity in its deck line. This statement would offer criticism of the commonly called "three-island type." It would seem likewise to favor the adoption, at least at present, of the flush-deck type as far as possible. The chief difficulty with the three-island type is the provision of longitudinal expansion in the bridge deck and in the sides of the vessel between the bridge and main decks.

The speed of a reinforced-concrete ship has thus far been assumed to be the same as a steel ship having the same outer form and dimensions. The only experience which we have at present with the surface friction of a concrete vessel or of concrete in water has been obtained from flow of water through pipes and conduits. It is common to assume that in the case of pipes and sewers, the surface of concrete presents a smoothness equal at least to the surface of steel. It is quite likely that with proper care in the surface of forms in which concrete vessels are cast, the outer surface may be materially smoother than the surface of a steel vessel.

MACHINERY ARRANGEMENT.

The machinery arrangement has an important bearing on the design of a concrete ship. Naval architects are familiar with the fact that very often with the engines aft in a cargo ship the hogging moment is larger than with them amidships. This may not be true with an oil tanker if the oil compartments are grouped amidships to neutralize the hogging effect of the engines aft. It is more desirable to have a larger hogging moment than sagging moment, because such an arrangement will reduce the likelihood of cracking below the waterline. The location of the machinery, therefore, has a large bearing on the design of the strength of the ship because of its influence on the disposition of the cargo.

In every phase of the design of the concrete ship, the principle of continuity cannot be over-emphasized. The path of stress should be as direct as possible, and there is ample evidence that sudden change of section of the member carrying the stress results in failure of varying magnitude. The transfer of stresses around openings in the deck must require far more attention than would similar ones in the floor of a building. The design of the connection of either end of any deck erection likewise requires the greatest care.

HOW TO COMPUTE LONGITUDINAL STRENGTH.

The data necessary for the computation of the longitudinal strength of a ship are derived from computations well known to naval architects. It may be well, however, to rehearse the operations gone through in determining the bending moments and shears which act upon the vessel.

It is necessary, first of all, to make a preliminary design of the vessel at hand, having previously determined the proportions from stability computations, and to make a close estimate of the weight of the vessel, all its appurtenances, and its probable cargo. It is convenient to lay off the length of the vessel between perpendiculars, that is, the length of the vessel on the water line from the forward face of the stern post to the after face of the rudder post to such a scale as will cause the plotted length to be 40 in. The purpose of so laying the vessel off is to reduce all forms and lengths to a common base length in the graph so that direct comparisons may be made with regard to different forms, different sizes and different interior arrangements of vessels.

Using this as a base line, it is next customary to lay off as ordinates normal thereto the weight in tons per foot of vessel to some convenient scale. The irregular line in Fig. 1, connecting the extremities of these ordinates, is called the "weight curve." The area between the base line and this weight curve is, therefore, with due consideration to scale, the displacement of the vessel times its length. The center of gravity of this area will mark the fore-and-aft center of gravity of the vessel. These weight curves are constructed for different conditions of cargo content, fuel content and ballast-water or trim-water content.

The Bonjean curves may next be constructed for the vessel. These curves, also shown in Fig. 1, are constructed by the draft to a given water line as an ordinate and the corresponding cross-sectional area of the vessel below each correspondingly successive water line at the particular station for which the curve is drawn. These Bonjean curves are constructed at various intervals or stations throughout the length of the vessel. The particular wave used in longitudinal strength computations is the trochoidal wave. Its length between crests is taken as the length of the vessel between perpendiculars, and the height of the crest above the trough is taken as one-twentieth of the wave length. The wave profile, therefore, is a trochoid.

Some naval architects prefer using a wave which has a height of other than one-twentieth of the length. Some use a tenth, some a fifteenth, and some a twenty-fifth. It is believed that for vessels of merchant size, however, a length of twenty times the height is the most common.

There are two conditions which a ship must undergo in passing over waves: (1) It must ride the crest of the wave with its forward and after end over a trough, which condition of strain is called "hogging." (2) It must rest with its two ends at crest of the waves and with its midship section over the trough, which condition of strain is called "sagging." It will readily be recognized that these two conditions give bending moments and shears of opposite signs. We will here describe the determination of the first in a vessel during hogging, that is, with the vessel astride a wave crest.

The profile of the vessel is superimposed upon the wave profile with the crest approximately at the midship section. At each station in turn the depth of water measured from the keel up to the wave profile is noted, and the transverse section area for that depth of water read from the Bonjean curve for that station. This transverse section area is next plotted as an ordinate on a base line opposite the station considered. This operation is repeated for all of the stations upon the vessel profile and a smooth curve drawn between the extremities of all the ordinates. This resulting curve is the trial curve of buoyancy.

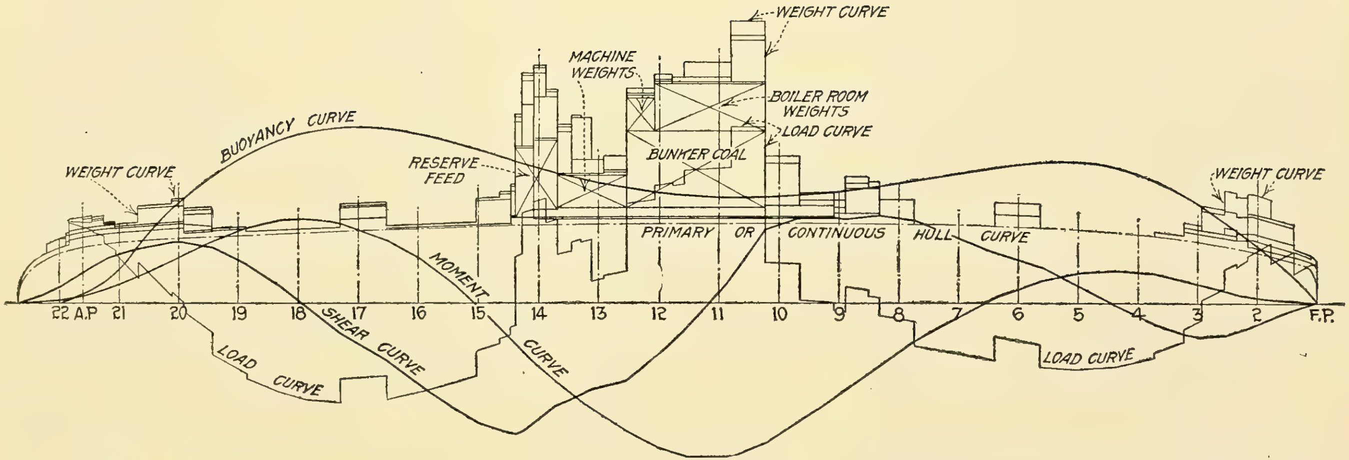


FIG. 1.—MOMENT AND SHEAR DIAGRAM OF GOVERNMENT 3500-TON CONCRETE SHIP, UNDER STANDARD SAGGING CONDITIONS AND LIGHT LOAD.

It is necessary to test this trial curve of buoyancy in two ways: first, the center of the area under the buoyancy curve must lie in the same vertical plane as the center of gravity under the area of the weight curve; and second, the area under the buoyancy curve must equal the area under the weight curve, that is to say, the upward pressure of the water must equal the downward load. If either one or the other of these two conditions is found to be in error, the trial curve should be corrected until a buoyancy curve is found which will satisfy the conditions. The final buoyancy curve may now be superimposed upon the weight curve as shown in Fig. 1. We may now, at each station of the vessel and at as many other points as convenient, determine the difference in ordinates between the buoyancy curve and the weight curve. This difference may be plotted as a resultant ordinate from the base line, giving the curve, which is known as the "load curve."

Ordinates to this curve represent the actual resultant load acting upon the vessel. It now remains only to integrate either analytically or graphically this load curve to obtain the shear curve and in turn to integrate the shear curve to obtain the bending moment, Fig. 1. The operation is repeated for the condition of sagging. Each of these operations is repeated for each condition of loading which might prove to exert a severe strain upon the vessel.

After these graphical constructions have been made for determining the bending moment and shears, it will be possible to pick out the maximum value for shear and the maximum value for bending moment for both hogging and sagging for various conditions of loading of the vessel. The maximum value for bending moment may be expressed in the form of

$$\frac{\text{total load displacement in tons multiplied by the length in feet}}{\text{coefficient}}$$

and the maximum value for shear may be expressed in the form of

$$\frac{\text{total load displacement}}{\text{coefficient}}.$$

As an approximate check it may be noted that the maximum bending moment will have a coefficient of approximately 35 to 40 and the maximum shear will have a coefficient of 13 or 14. It might be here noted further that in barges and small craft where concentrated or intermittent loads are possible, the coefficient for bending moment may be as low as 20 in some cases and for shear as low as 7½. Barges, scows and small harbor craft need special consideration with respect to manner of loading which would not be properly expected in seagoing craft, because of the possible danger of shifting cargo during rolling and pitching.

The bending moment which is found as a maximum at a point in the midship section is considered to act over 40 per cent of the middle body instead of acting as the exact form of the bending moment curve for hogging and sagging would indicate. The steel is carried forward fore-and-aft of the midship section to provide for this moment.

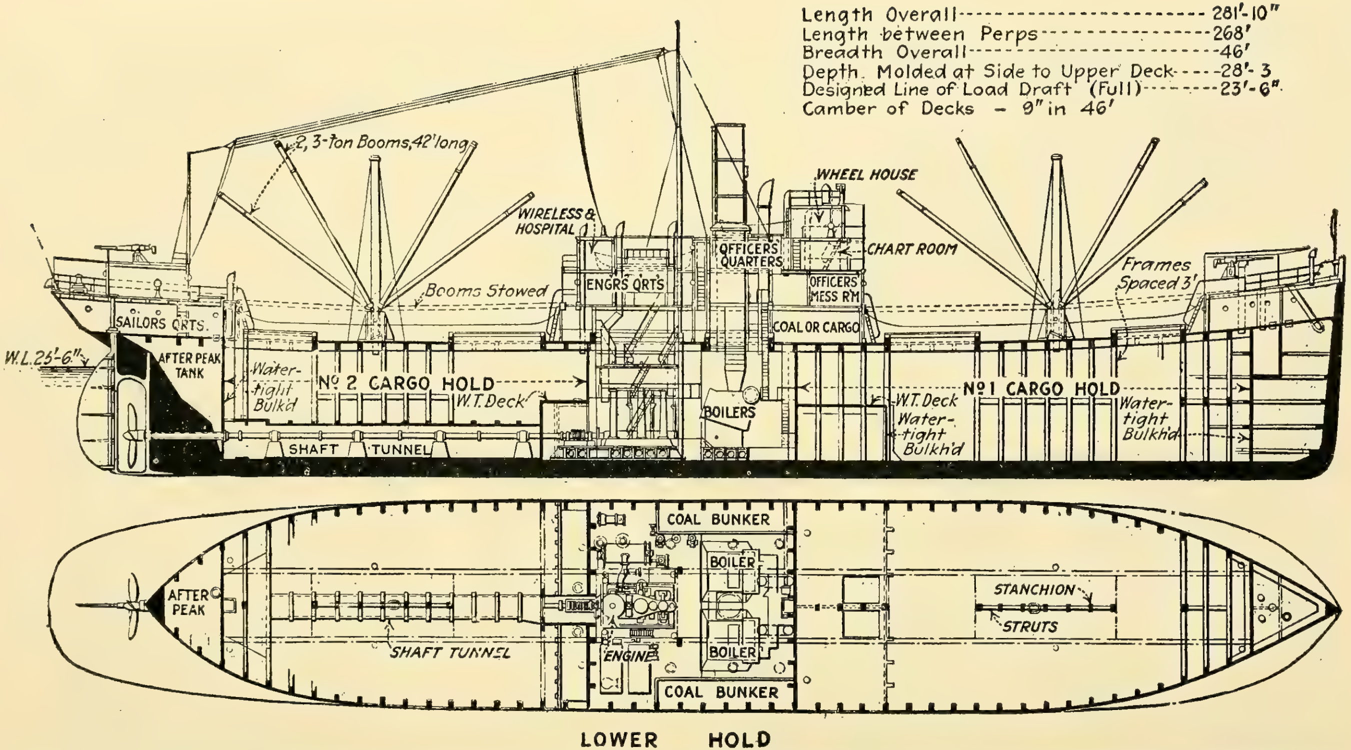


FIG. 2.—IN GENERAL LAYOUT THE GOVERNMENT'S NEW 3500-TON CONCRETE SHIP IS SIMILAR TO THE STANDARD WOODEN SHIP BEING BUILT BY THE EMERGENCY FLEET CORPORATION.

Having now the attacking bending moment and shears, it is necessary to compute the properties of the cross-sections at the points of maximum moment and maximum shear. Properties of the section which are important are the position of the neutral axes, the moments of inertia in hogging and sagging, the arms of the resisting couples commonly known as *jd*. It is common to neglect tension in the concrete in making these computations. The location of the neutral axes may readily be determined by tabular computations, and at the same time the moments of inertia about the base line may be determined and later transformed to the neutral axes. In a similar manner the values of *jd* may be determined.

Section modulus may now be computed and the fiber stresses at any point in the cross section determined for the various bending moments. Likewise the unit shear in the side walls at the neutral axes may be determined from the common formula V/bjd. It is very instructive to construct a curve of shear displacement for the total cross section of the vessel. Such a curve will point out the fact that the shear in the side walls of a vessel is very nearly constant between decks or between the bilge and the next deck above.

The flexural stresses in the gunwales and in the bilges are greater when the vessel is listed than when righted. The neutral axis in the listed position may be very approximately located from the fact that neutral axis is that diameter of the central ellipse of inertia of the cross-section which is conjugate to the trace of the plane of the attacking moment. One need, therefore, only to construct the axis of the ellipse of inertia, having previously determined the moments of inertia about both axes, to next lay off the trace of the plane of the attacking moments (for instance, if the vessel is listed 30° the plane of the attacking moment would be 30° from the vertical axis of the cross-section); and lastly to determine the diameter conjugate to the trace of the plane of the attacking moment. This conjugate diameter is the approximate position of the neutral axes, being slightly in error because of the fact that during listing the shifting of the neutral axis permitted tension to exist on one side of the neutral axis on one side of the vessel and neglected the compression adjacent to the neutral axis on the opposite side of the vessel. It is possible to make the adjustment if the shifting of the neutral axis involves the inclusion or exclusion of a considerable area of material. Having found the new position of the neutral axes after listing, the corresponding moment of inertia may readily be found analytically if desired, upon a consideration of the extent of angular transfer.

The section modulus may then be computed and the stress determined for the particular position of list. The attacking bending moment in a listed position is commonly assumed to be the same as for a righted position. The difference in stress between the listed and righted positions shows an increase of from 10 to 20 per cent for the listed position over the righted position, depending upon the form of cross-section. It has been the practice of the Department of Concrete Ship Construction to use an extreme maximum fiber stress in the concrete of 1500 lb. per sq. in. The stress in the steel below the water line for combined longitudinal flexural and local stresses is taken as 12,000 lb. per sq. in. and the similar combined stresses in the deck reinforcement is taken as 16,000 lb. per sq. in.

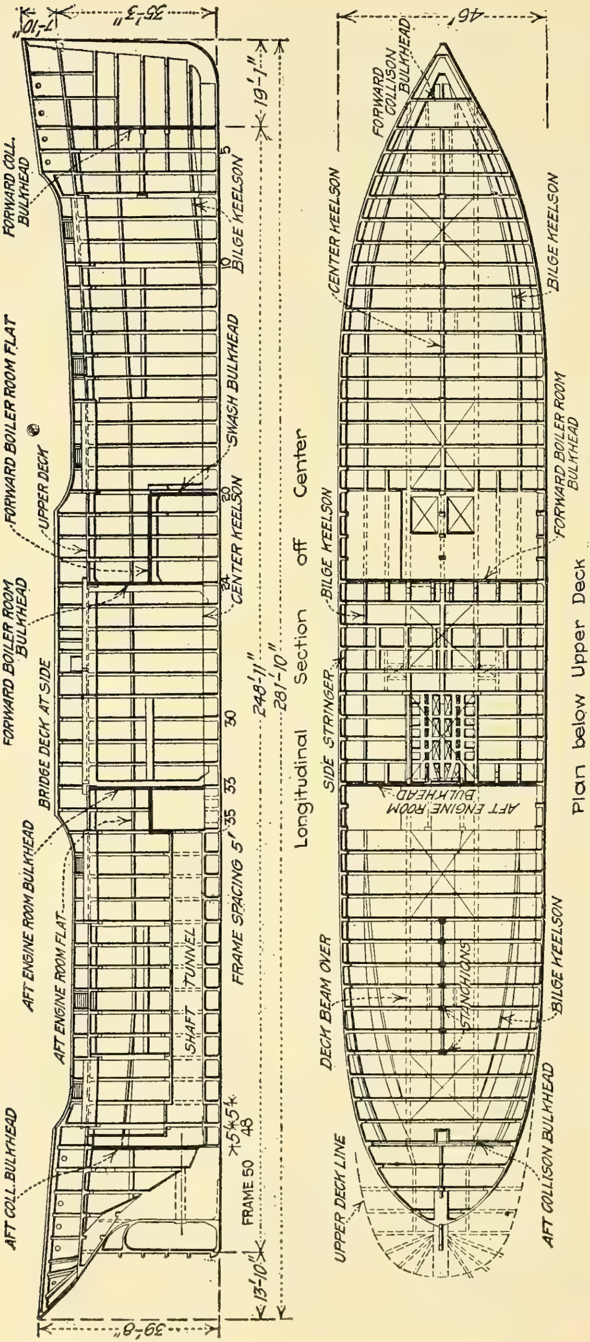


FIG. 3.—CONCRETE FRAMING IN 3500-TON CONCRETE SHIP. ADAPTATION OF SIMPLE BEAM AND SLAB DESIGN.

DISTRIBUTING STEEL IN THE SECTION.

It will be noted on reviewing the above procedure in determining the properties of the ship's cross-section, that a steel distribution and the concrete form have to be assumed before computation can be made. To this end it is possible to approximate the requisite amount of steel by roughly computing the steel required as for an ordinary concrete beam, assuming j to be 0.93 with an approximate value of jd. An approximate steel area may be assigned to the bottom and to the deck before beginning more detailed computations. There has been considerable discussion concerning the ability of longitudinal steel in the deck and bottom which lies approximately on the center line to develop stress during flexure of the vessel. In ordinary practice the nearest analogy we have to the action of this steel is that action which occurs in ribbed floors in which the ribs are thirty or more thicknesses of the slab apart. The slab in such a ribbed floor is in compression when there is flexure in the ribs and the unit compression stress is higher immediately over the ribs than midway between the ribs and parallel thereto. The same action occurs in an I-section in flexure when the leg of the outstanding flange is 30 or 40 times its thickness. In both cases, and more frequently in the latter perhaps, curling results on the transverse section near the end. To what extent this action may be found in a reinforced concrete vessel has yet to be determined. Undoubtedly, if the stress is smaller near the inboard than near the outboard, the stress outboard must be higher than that originally intended in order to develop the total tensile resultant necessary to maintain the resisting couple. Now if it were possible to determine the stress in the deck or in the bottom of the vessel's centerline and also the stress in the deck or in the bottom near the outer wall, it would then be possible to compute the longitudinal shear stress between the center line of the vessel and the outer wall. Even though this stress on the center line may be zero, the longitudinal shear in the deck and in the bottom near the side wall would not be as large as it would be in the side wall near the deck and in the bottom respectively.

There are many difficulties which are met with in the distribution of the longitudinal steel in the cross-section of the vessel. It seems reasonable to suppose that the most effective position for the longitudinal steel would be as near the sides as possible; or in the case of vessels with longitudinal bulkheads as near the sides and the bulkheads as possible. This merely groups the steel close to the members which take the shear.

If one studies the dimensions of various sizes of ships, he may readily realize that the depth of the ship does not increase in proportion to the increase in bending moments for an increasing carrying capacity. This means that in the larger sizes of vessels the flange steel increases in greater percentage than for smaller sized vessels and this fact points to the serious difficulty of disposing of all of the steel required in the positions noted above as desirable places for grouping the steel rods. Further complications arise in stresses produced by local flexure due to the external water pressure and just above the bilge to shear in the sides. Different types of framing distribute the stresses differently, but it is logical to suppose that that system which most nearly balances the stresses in the various sections is the system which is most economical and durable from the standpoint of structural strength.

The maximum shears acting upon the vessel occur at approximately the quarter points of its length. Both for hogging and sagging it will be found that at a point near amidships the shear will become zero. As the vessel passes from hogging to sagging it will be found that the shear may not be zero at the center of the length of the ship as given by these two cases but may be in some instances two-thirds as much as the maximum shear. It would seem consistent practice to carry the scheme for the shear reinforcement continuous between the quarter points.

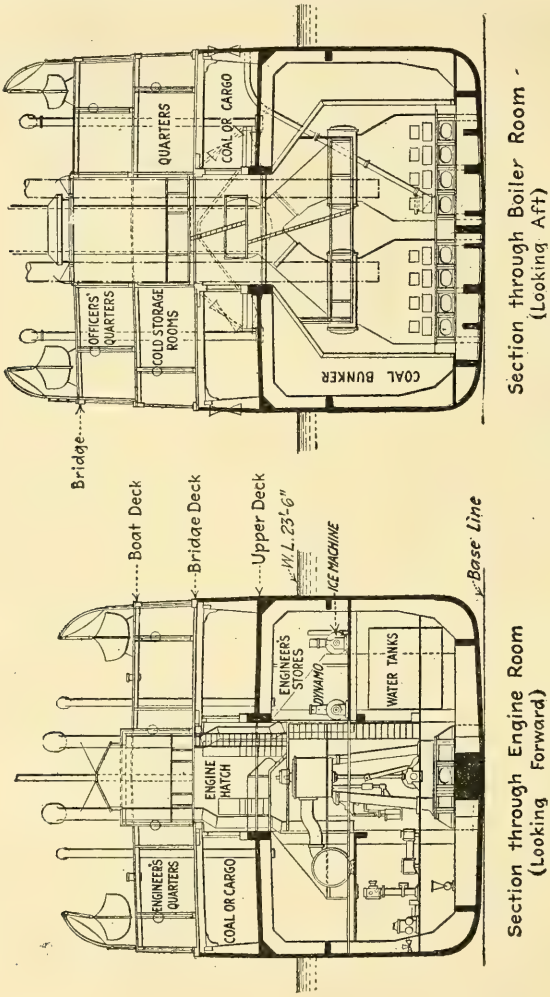


FIG. 4.—TWO CROSS-SECTIONS OF THE GOVERNMENT'S 3500-TON CONCRETE SHIP.

SYSTEM OF REINFORCEMENT.

Two systems of reinforcement may be employed, one in which the reinforcement runs diagonally and another in which the reinforcement runs vertically and horizontally. There are many practical considerations which require study before the choice may be made between these two systems of reinforcement. These considerations will be pointed out in considerable detail in the discussion to follow concerning types of framing of vessels.

The shearing unit stress which is used in the design must, if possible, be much higher than that used in building practice in order to obtain a vessel of economic weight. Early designs made by the department were based upon the unit shearing stress of 300 lb. per sq. in. A later design was raised to 400 lb. per sq. in. and the department is at present engaged on a design of an oil tanker of 7500 tons D. W. capacity, using approximately 500 lb. per sq. in. in shear.

The department has conducted some tests at the Bureau of Standards Laboratory at Pittsburgh, under the direction of Prof. W. A. Slater of the University of Illinois. Although these tests do not by any means cover the field of ship construction, they do justify the use of the shearing values above mentioned, provided that the unit stress in the steel in a tensile direction be assigned the proper value. For instance, if when designing with a unit shear stress of 500 lb. per sq. in. the vertical shear bars are obliged to carry all the shear at 10,000 lb. per sq. in. in tension, the combination will be worked at a factor of safety of approximately 2.5. From the tests at hand it seems reasonable to limit the tensile stress in vertical bars to 10,000 lb. per sq. in. and to limit the tensile stress in the diagonal bars to 16,000 lb. per sq. in. It is possible with these unit stresses to use the formulas proposed by the Joint Committee for the design of members in shear.

The design is usually made to meet the shearing stresses in the sides and if the shearing reinforcement is carried around the bilge and gunwale into the bottom and the deck, it would be expected that the shearing stresses in the bottom and in the deck were amply provided for. This is the practice at present in use by the department.

Cracking will occur in the use of shearing values and steel stresses mentioned above if these stresses are developed. These cracks, however, are incipient and are not calculated to admit water even under considerable head. This is being investigated to determine approximately the size of crack above which seepage may be expected. It is believed, however, that such cracks will not interfere with the range of unit stresses mentioned above.

TRANSVERSE STRENGTH.

The transverse strength of a vessel is usually supplied in wood and steel by ribs or frames. The same provision for strength has been maintained in the concrete vessel. If the frame type is used the water pressure against the hull is transmitted immediately to closely spaced transverse frames. On the other hand, if the Isherwood system is used, the water pressure against the shell is transferred to widely spaced transverse frames through the action of closely spaced longitudinal stringers. These frames may take various forms to withstand the pressure against the ship, such as an open rectangular frame; an arched frame; a frame with intermediate hold-beams or decks and stanchions, and frames with some form of trussed bracing.

Practice in commercial naval architecture is at present to proportion the size of the transverse ribs and other members by data based upon previous experience with various weights of frames. The rule books of the registry societies for instance often form the basis of a design of a steel or wood merchant ship, the size of frame members being taken directly from the tables therein.

Two methods are followed at present in the design of frames for concrete ships. The first relies upon the rule book to determine the size of a steel frame for a vessel of similar dimensions and then duplicates this steel frame in concrete. The second method assumes possible exterior water pressure or interior cargo pressure, or combinations of both, and analyzes the frame according to some phase of the elastic theory. This insures the determination of the bending moments and shears developed under the loads applied which may be cared for by properly designing the members to resist them. If the transverse frame is formed of curved members for the most part, it is analyzed by some one of the various arch theories. If the members, however, are arranged in some form of polygon, the method of slope-deflections may be found to give a ready solution. An ingenious treatment of slope-deflections was independently developed by G. A. Maney and published by him in Bulletin No. 1, Engineering Studies, University of Minnesota, in March, 1915. As an approximate check on results comparison may be made to steel vessels of known seaworthiness and similar size.

There are many advantages and disadvantages in each of the types of interior framing at present proposed or in use. The Isherwood and frame types are the most common and have received the most consideration. The Isherwood system provided for its local stresses in the slab by rods running around the ship's girth and hence normal to the section of longitudinal stress. The ribs running between the wider-spaced frames, however, are subjected to bending, which bending is in the section of the longitudinal stress and therefore must be considered in making the combination for maximum stress. The frame type, on the other hand, reinforces against local stresses in the slab by means of rods running longitudinally in the shell. It may be readily seen that this fact reduces the efficiency of the longitudinal steel because of its effect in lowering the component tension which is available for resisting longitudinal flexure. It is claimed by some that the frame type is an inferior system when hogging or sagging because of the absence of longitudinals between frames to assist in carrying the compression.

TORSIONAL STRENGTH.

Torsional strains in the vessel are the most difficult to compute. The stresses set up are essentially shearing stresses which must be carried by the shell of the ship. Since the common form of vessel is wider than it is deep;

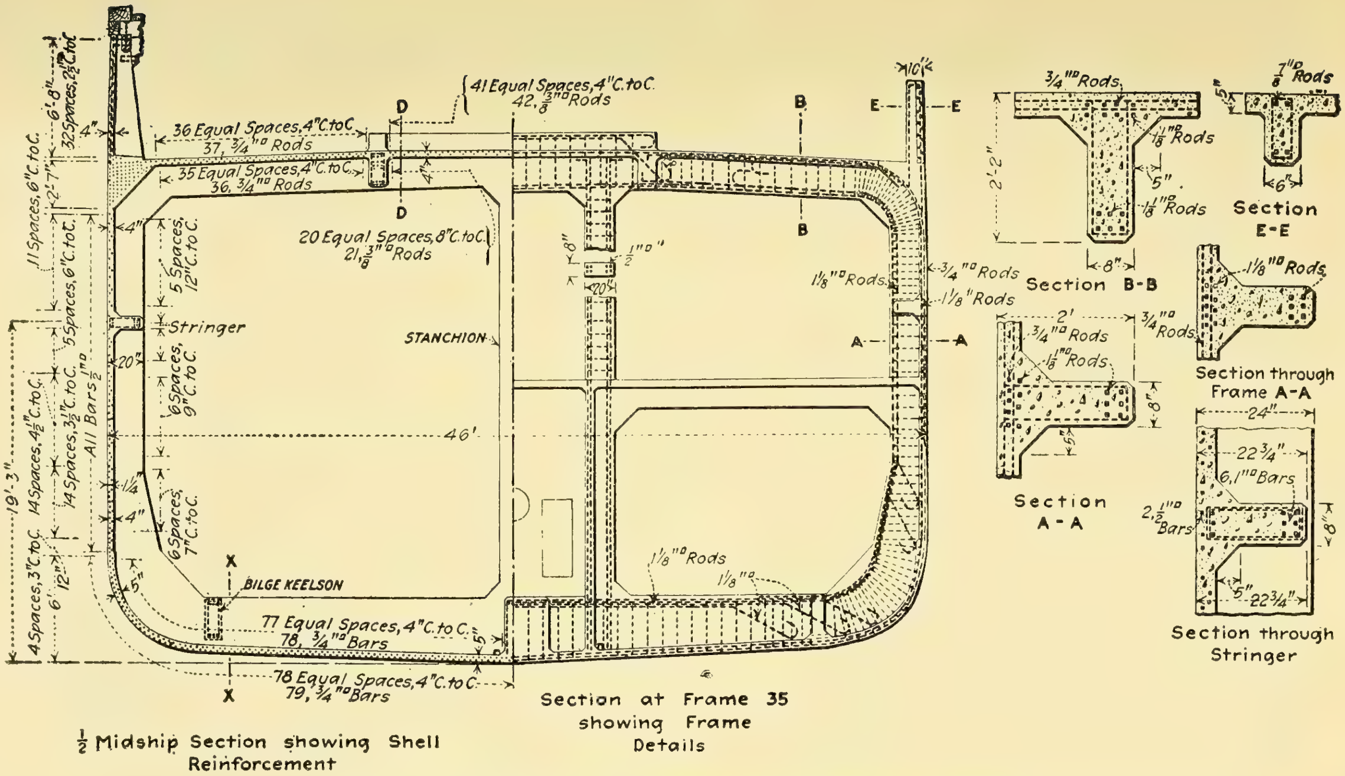


FIG. 5.—CROSS-SECTION OF SHIPPING BOARD'S 3500-TON CONCRETE SHIP.

the maximum torsional shear is to be found in the deck and bottom, at the center line of the vessel. Because of the openings necessary in the deck the torsion produces the most severe results near the midship section in the neighborhood of the hatch openings, particularly at their corners. The torsional moment will not exceed a fourth of the loaded displacement of the vessel multiplied by its maximum righting arm. This will be somewhat excessive for the larger vessels now under design. The maximum torsional stress will not occur simultaneously with the maximum longitudinal stresses, because the vessel must be in a quartering seaway, with the wave period shorter than the vessel's length.

After the main computations have been made upon the requisite strength of the vessel, a vast amount of detailing of the members remains to be done. All attachments of fittings to the hull must be provided for and carefully detailed after requiring a rearrangement of the structural reinforcement. Drawings showing the arrangement of the rods in the members are first made, as is common in concrete work, after which bending diagrams are made for each separate form of rod. In fact, the greater portion of the steel in concrete vessels encounters a curve of some form. It may, therefore, be readily appreciated the amount of detailing necessary to furnish requisite working information to the shop and in the field.

Details of equipment are no different than similar details for vessels constructed of other material, but the method of attachment employed with each separate fitting is very often far different from the method of attachment employed in steel or wood ships. Special care must be exercised in placing any castings which penetrates the outer shell or any casting which passes through any watertight bulkhead. Many of the attachments on the deck and in the interior of the vessel may be made by molding the anchor bars permanently in concrete at the time of pouring. Any fitting, however, which is apt to require replacement should be so attached as to be removable for repair or renewal. The most difficult attachment of all is the cast-steel stern frame of the vessel. This frame supports the rudder and the after end of the stern tube so that it is required to take the lateral thrust of the rudder and the vibration of the propeller. The shoe of the stern frame which connects the lower end of the rudder stock at the point of juncture of the keel with the lower end of the stern post is so designed as to be able to withstand the riding of the vessel on a bar or other obstruction. Particular attention is paid to the ease with which the stern frame may be repaired if once broken. It is probable that a large percentage of breakage to stern frames on concrete vessels will occur in the shoe or in the lower end of the rudder stock. This portion of the stern frame should be so scarphed as to be removable for repairs. The frame may be designed so that the whole stern frame may be detached from the hull. This may be accomplished by putting on cheek plates which extend from either side of the stern frame casting back along the side of the skag or surface of the ship at the stern. The reinforcing rods may be rigidly attached in a number of ways to the inner face of these plates. In some instances these plates may be cast as flanges from the stern frame and the same method of attachment of the reinforcing bars effected.

COMPOSITE DESIGNS.

Many attempts have been made to produce a satisfactory design in which part of the vessel is structural steel and the remainder reinforced concrete. In developing a design of this kind there are two fundamental principles to be borne in mind; first, there must be the proper proportion of steel and reinforced concrete to produce unity of structural action of the two materials; and secondly, an adequate connection must be made between the two materials.

Many concrete men think of a steel frame as carrying the load, and any concrete applied thereto is thought of as an independent, unstressed unit. This is of course contrary to structural principles. The materials will take the load in proportion to their respective rigidities. In ship analysis, involving as it does more structural problems than any other structure which the concrete engineer has been obliged to analyze, it is an exceedingly difficult matter to estimate the share of work done by the two materials.

Concerning attachments, it is frequently believed that the principal function is similar to the casting of floors and walls to a steel skeleton. Reversal of stress, grinding, chafing, change of section, and bearing should be borne in mind. It is not an impossibility to develop a satisfactory attachment; but it is the general feeling that the use of two materials in this manner is not so reliable as the use of a single material and is really more experimental than an all reinforced-concrete structure.

CONSTRUCTION PROBLEMS.

The construction problems in every phase are no less difficult than those of design. We are in effect carrying out construction work of laboratory quality on a field scale. Think of building walls 4 in. thick, over 400 ft. in length and 35 ft. in height, without defect of any kind. We must build not one of these walls, but many of them, for there are two walls in each cargo ship and four in each oil tanker. But the construction of the walls themselves is only a small part of the work, for there are innumerable inserts and fittings to be attached, several layers of steel to be exactly located, and the forming of the intersecting frames and bulkheads, foundations for engines, boilers, and all other equipment. These elements do not involve simply the assembly of normal building methods and material to a ship, but in almost every detail require greater refinement and care.

Materials.—The first thought of most engineers with regard to materials for the concrete for a ship is to use such as will provide the densest mixture possible. Concrete of high density also has a high unit weight. Our problem is four-fold: The concrete must be durable and impermeable to water; it must have a compressive strength of at least 4000 lb. per sq. in. at 28 days, and a minimum weight. To meet these conditions we have determined upon the use of a rich mortar mixture.

Any standard portland cement which will meet the specifications of the United States Government may be used, provided the fineness is increased so that at least 90 per cent will pass a No. 200 sieve. The greater fineness is required in order to obtain greater strength and a more plastic mixture. Such a cement is also more volume constant.

For the present the maximum size aggregate is limited to ½ in. A sand and gravel or a volcanic ash or specially burned clay may be used. The commercial future of the concrete ships is in large measure dependent upon obtaining a light weight concrete. Our investigations are quite promising. The strength and weight of one of these mixtures which meets our requirements is as follows:

One part cement to one part special fused clay below ¼ in. size to two parts same aggregate between ¼ and ½ in. size had a compressive strength of 3380 lb. per sq. in. at 7 days and 4350 lb. per sq. in. at 28 days. It weighed 106 lb. per cu. ft. in a saturated condition.

With the use of this material the ratio of the dead-weight to total displacement will be 62 per cent for the 3500 ton ship as compared with 65 to 68 per cent for a steel ship and 53 per cent for a wood ship.

No integral waterproofing compounds of any kind will be used in the mixtures.

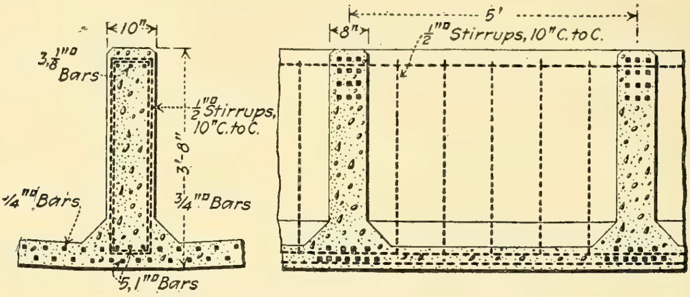


FIG. 6.—SECTIONS OF CENTER KEELSON.

Reinforcing steel which we are using, other than fabric, is of rods or bars rolled from new billets to conform to the American Society for Testing Materials Standard Specifications for Structural Grade New Billet Steel. Plain round bars will be much easier to fabricate than deformed bars, but on account of the uncertainty of the effect of reversal of stress we are using deformed bars wherever the bond stress is high.

Forms.—The form problem is a most intricate and difficult one. The forms for practically the entire ship must be placed before concreting starts. The accuracy of placing must be much greater than in ordinary building construction, for if the spacing is slightly oversized, it results in increased tare weight of the hull, which, of course, decreases the cargo capacity of the ship. The relatively high unit stresses and the small thickness of covering for the steel also requires unusual accuracy in placing the forms. There are a number of ways of constructing the forms. They may be of wood or metal, or of wood lined with metal. It has also been suggested that plaster forms may be used. They may be built in panels or as one unit, to be later removed, board by board. The inside forms may be first constructed and the outside forms placed after fabricating the steel. A more common method is to place the outside forms first, fabricate the steel, and then place the inside forms. The former method is being followed in the construction of one ship for the Emergency Fleet Corporation, but the latter method will be used on subsequent ships. Much time and money can be lost or saved in forming, and much thought and study can profitably be given to this work. In Norway there is one company building small craft by the so-called "upside-down" method, in which the inside forms are fabricated in an inverted position. The concrete is shot into place with a gun, or applied by hand, a relatively stiff mixture being used. No ships are being built by this method in the United States.

Reinforcement.—The accurate bending of the reinforcing steel is one of the most difficult of all the construction problems. With a wall thickness of only 3½ to 4 in., and two to three layers of steel, you can readily appreciate the need for accurate bending of the steel. Furthermore, the curvature is constantly changing forward and aft in the vessel, which requires constant change in the location of the dogs on the bending table. The use of small steel is recommended, in that it is easier to spring into place if the bending is not accurately done.

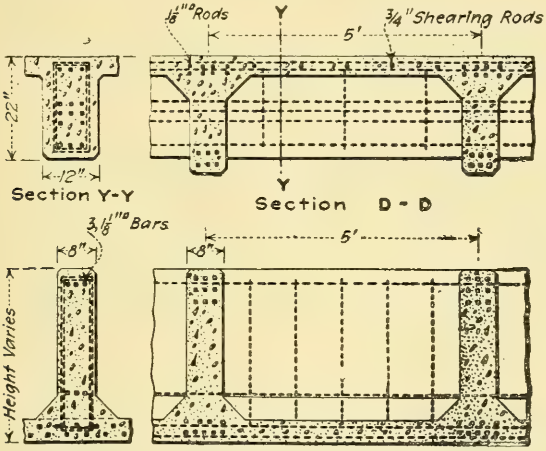


FIG. 7.—SECTIONS THROUGH SHIP SHOWN IN FIG. 5.

With a very large quantity of steel approximately equivalent to 10 per cent of the carrying capacity of the ship, to be placed in a very small space in thin walls, there is much opportunity for the development of ingenious methods of fabricating the steel. Undoubtedly much time can be saved by pre-fabricating, at least in part, the steel of the frames and placing them in the ship as units. These frames may be as much as 54 ft. in width, and 35 ft. in height, and they must be accurate to within a fraction of an inch. Through a length of 60 per cent of the ship the shape of these frames is constantly changing. All the steel must be supported and secured so that it will not touch the form surface. Numerous methods have been suggested and there are a number of chairs now on the market for supporting steel, but all of them have objectionable features.

Welding.—A study has been made of welding methods and machines as a means for avoiding the lapping of the steel. The acetylene weld is rather uncertain and not satisfactory. The electric resistance weld appears to be the most promising. Several machines of this type have been ordered and are now being tried out. The bond stresses are exceedingly low in the longitudinal steel and there is more space available for lapping. In the frame steel, however, it is so close together there is not room for lapping without increasing the thickness of some of the concrete members, which will in turn increase the tare weight of the hull. The weld by the electric resistance method can be made quite rapidly, and at small cost. We do not anticipate welding any steel other than the main steel of the frames and possibly the main longitudinal steel.

Rich mortar mixtures will be used and they must be carefully proportioned and mixed in order to insure the quality of concrete necessary. Special attention should be given to the selection of the mixer as all common types of mixers will not thoroughly mix mortars. The concrete should not be transported from the mixer and deposited directly into the forms in continuous operation or in large batches on account of the danger of not having it thoroughly worked into place about the reinforcing steel. We are for the present requiring that all concrete shall be shoveled into the forms in order to insure its deposit in small batches and its thorough working into place.

The actual placing of the concrete is a small item in the cost of construction of a ship and therefore there should not be the incentive to rush the work as in ordinary building construction.

CONCRETE.

It is preferable to place the concrete as one continuous operation in order to avoid construction joints. This will require approximately 3 days (of 24 hours each) for a 3500 ton ship and 6 days for a 7500 ton ship. While it may appear to some of you that it will be difficult to work the concrete thoroughly around and through the reinforcing, we do not anticipate any trouble from this source. A rich mortar mixture which we propose to use is quite fluid, even though it is not mixed to a wet consistency, and with a slight tapping of the forms, it readily settles into place about the steel. A number of test panels have been made, and the results in all cases have been entirely satisfactory. It is very difficult to make a construction joint in a section in which is embedded a large quantity of steel. Most leakage troubles occur at construction joints. The uncertainty of the bond obtained at construction joints is also objectionable on account of the large shear stresses. Several mechanical methods of placing mortar and concrete have been given consideration but the results of our investigations do not up to the present warrant their use.

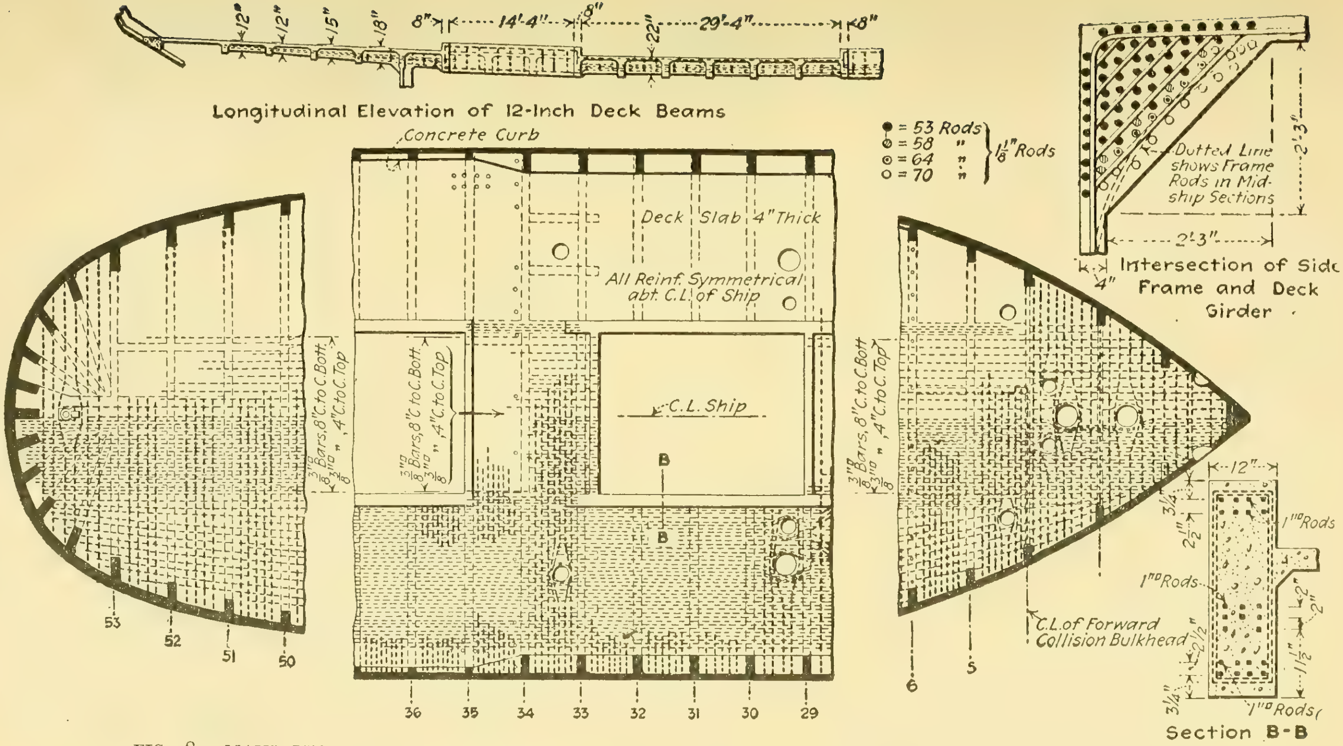


FIG. 8.—MAIN DECK REINFORCEMENT AND DETAILS SIMILAR TO ORDINARY REINFORCED-CONCRETE FLOOR.

TREATMENT OF HULL SURFACES.

It has been suggested by some that rough lumber be used for the outside forms and a plaster coat be applied to the surface of the cast concrete. The application of such a plaster coat is not recommended for the following reasons:

The surface to be covered is very large, and the application of such a coat would have to be completed in a very short time. There is absolutely no way of positively assuring a perfect bond between the plaster coat and the concrete surface. The application of such a coat will increase the weight of the hull without compensating advantages.

Barnacles and sea vegetation will adhere to concrete in the same manner as it develops on steel and wood. It is therefore necessary that the bottom of the hull be coated with some anti-fouling paint. The same paints can be used as are now used on steel ships except that special means must be taken to insure a thorough bond. Investigation of a number of different types of paint and coatings is now being made. It is quite probable for the present that a bituminous coating will be employed on the bottom of the government ships.

LAUNCHING.

There has been much discussion in our organization relative to the advantages and disadvantages of constructing the ship on ways which will permit of side-launching or end-launching. Ways built for side-launching permit of the construction of the ship on an even keel, thus making all the lines plumb. There have been very few accidents by side-wise launching of steel or wood ships, and we believe the concrete ship can be safely launched somewhat earlier in this manner. In end-wise launching the conditions of sagging or hogging depend upon the accurate positioning of the ways, and determination of the weight and conditions of buoyancy of the ship just before it leaves the ways. Slight settlement of any of the way piles may cause very serious hogging or sagging stresses. Two of the vessels being built by the Emergency Fleet Corporation will be launched end-wise, and the others will be launched side-ways.

INSTALLATION OF EQUIPMENT AND MACHINERY.

Certain of the equipment, such as the hawse pipe, stern frame, stern tube, etc., are cast into the concrete. The major portion of the equipment and all the machinery is secured to the concrete by anchor bolts. These must be accurately placed in the forms, and completely detailed before the concrete is poured. Aside from the connections, the installation of machinery and equipment does not differ from the conditions met in either wood or steel ships. Wherever possible we recommend the use of through bolts rather than anchor bolts. Under all deck machinery and fittings suitable waterproof gaskets should be placed so that there will be no leakage around the holding down bolts. Too much attention cannot be given to the careful detailing of the fastenings and their exact location in the hull.

DURABILITY.

If durability is to be obtained, special attention must be given to many elements of the ship. The most serious problem is to provide means of adequately protecting the steel from corrosion. There is a large quantity of steel embedded in the concrete and much of it cannot be covered by more than $\frac{5}{8}$ in. of mortar. This, of itself, we know will not protect the concrete, particularly in the interior and in the upper portions of the hull. There are two means of allaying, if not wholly preventing, the corrosion of the steel. The steel may be galvanized or painted with some protecting medium which will not appreciably affect the bond, or the concrete may be coated with some thoroughly impervious membrane which will prevent both the air and water from reaching the steel. A large number of tests are being made and it is quite probable both methods of protection will be tried. The results are promising and we believe a satisfactory protection will be developed.

Another possible disintegrating element which may have great importance is the effect of constant reversal of stress, as the ship alternately is subjected to hogging and sagging stresses in a heavy sea. Our allowable steel stresses are such as to cause the concrete to crack. There are few analogous structures on land to which we can refer for guidance on this subject and only experience can tell what may be expected. Reversal of stress tests are now under way and we hope soon to have results for our guidance.

We do not anticipate any trouble from chemical disintegration except as the hull may be seriously abraded.

We estimate the life of the concrete ship without any special protection at several years and known methods which can now be applied should extend the life several years longer. We believe adequate protection will be developed to ensure reasonable permanent life to the concrete ship.

ELECTROLYSIS.

The use of direct current on a concrete ship should be avoided wherever possible, on account of the possibility of electrolysis from stray current. A very small leakage for an appreciable time, as demonstrated by the experiments of the Bureau of Standards, will cause a weakening of the mortar at the cathode, thus decreasing the bond strength. If the current leakage is large it may cause an oxidation of the steel at the anode, which will result in splitting and spalling of the concrete. The use of brass or bronze castings adjoining the concrete reinforcing should also be given consideration, on account of the possibility of local electrolitic action in the presence of the electrolite.

DISINTEGRATION OF CONCRETE FROM CARGO MATERIALS.

With the quality of concrete which we are using we believe no special provision will need to be made to hold the heavier fuel oils. We are, however, investigating various coating materials with a view to their use as a lining if such is found desirable.

Sugar and certain vegetable oils, such as the cocoanut and peanut oil, will disintegrate the concrete if exposed for an appreciable length of time.

If the ships are to be used in this class of trade, a paint coating should be applied to the inner surface of the hull, as a protective means.

The economy of the concrete ship cannot be determined until we have had experience over a term of years. The estimated cost of the concrete ship complete at the present time is approximately $125 per ton dead weight. The cost of a steel ship averages about $200 per ton, and that of a wood ship, $165 per ton deadweight.

If proper coatings can be developed to prevent deterioration, the concrete ship should be a competitor of the steel ship. With further experience it is believed the weight of the concrete can be very materially reduced, thus making the cargo capacity more equal to that of the steel ship.

Very extensive investigations are being conducted to obtain data which will permit more exact design, which will assist in increasing the efficiency of construction and provide means of protecting the ship under various exposures if such is found necessary. We are seeking information from every possible source and appreciate the help many engineers and others have rendered us when assistance has been requested.

The results of many of these investigations I hope may later be made available to you by the men who are devoting themselves to this work.

We have neglected to cover many points about which no doubt you wish information but the subject is too large and new to permit of complete detailed discussion at this time.

It is our hope that the concrete ship may remain after the war as one of its many blessings to civilization.

Committee Reports Presented to the 14th Annual Convention of the American Concrete Institute

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

The present abnormal conditions have interfered very seriously with your committee's work during the past year. This has been keenly regretted by the chairman, who at the time of the reorganization of the committee last fall, had reason to hope that some good work might be accomplished before this report became due. The opportunities that were in view at that time had to be given up, however, and it is only due to the interest of a few members of the committee that a program of work has been outlined and some progress made thereon.

It may be well to recall that two distinct fields of work are assigned to this committee, viz. the development of stucco specifications, and the study of concrete surface treatments. In former days stucco entered only as one form of concrete surface treatment, but as its use developed and the need increased for general stucco specifications, the work of the committee gradually became confined to this field, to the neglect of the broader one. In view of the present conditions, therefore, which are not favorable to the prosecution of tests that do not contribute to the winning of the war, the committee decided to lay a foundation for future work by the collection of data on existing stucco and concrete structures.

With reference to stucco, the committee has felt that although so-called standard specifications are now in existence, practice varies so widely in different parts of the country, and so little positive information is at hand as to the merits of different bases, mixtures, and methods of application, it seems advisable to go into the field and find out from actual observation what elements seem to be common to those stuccos which are satisfactory, and to account insofar as possible for the defects in those which are not satisfactory. This field work was originally planned as a part of the Bureau of Standards investigation of stuccos and plasters, but owing to pressure of emergency work in all Government bureaus since the beginning of the war, very little field work has been possible during the past year. However, as your committee has been as a whole closely in touch with the Bureau of Standards investigation, it has felt that its activities should supplement the Bureau's work, and supply as much of this field data as possible.

Preliminary observations have already been made on a limited number of stucco structures in the vicinity of Philadelphia, New York, Boston, Cleveland, Chicago, Pittsburgh, and Washington. The results of these observations are reassuring in that many stuccos were found to be in generally satisfactory condition after ten years or more of existence and apparently good for an indefinite time to come. Serious local deterioration that had occurred in some instances was attributable in practically all cases to accidental conditions, or to faults in design that could easily be corrected.

Briefly summarizing the results of this preliminary inspection, it may be said that stucco is extensively and successfully used; at the same time absolute perfection is not attained and there is much room for improvement both in structure and appearance. The experience gained in these observations has also demonstrated that inspections of stucco structures in which little is known of the histories of these structures is largely a waste of time, and therefore the first step in a properly conducted field investigation is the collection of fundamental data. For this purpose the committee has prepared forms which indicate briefly the nature of the information wanted, and which are being distributed as widely as possible. These forms provide for the following entries:

1. Type of structure.
2. Location.
3. Owner.
4. Architect.
5. Plastering contractor.
6. Estimated or approximate value of structure.
7. Date of plastering.
8. Base.
9. Stucco mixture.
10. Finish, color and texture.
11. Photographs.
12. Remarks.

Submitted by.................. Date..................
Address..

These records as they accumulate are being filed according to geographical location and as many of them as possible will be followed by careful inspections whenever opportunity offers. In this work the cooperation of the Plasterer's International Union has been obtained, and the committee feels that in this powerful agency it has the means of getting complete reports on stuccos in almost any part of the country. As the reports from this source will generally be furnished by experienced plasterers a series of questions has been prepared in terms of the trade, and are so expressed as to require the shortest possible answers. The forms on which these questions are submitted provide spaces for the answers after each question, and while they are not in final shape, they have been found by actual trial to cover the ground very satisfactorily. These questions are listed below to indicate the information desired in an inspection of this sort, and it is believed that the majority of them will be intelligible to the average home owner.

FIELD INSPECTION OF STUCCOS.

A report is desired on the condition of the stucco job at..................

....

....

Please answer the following questions relating to this job as fully as possible:

1. For what is the building used?
2. How many stories high is it?
3. How many feet frontage does it have?
4. Is it a corner building?
5. How does it face, north, east, south or west?
6. Who was the architect?
7. Who was the stucco contractor?
8. How much did the building cost?
9. When was it stuccoed? (Give month and year).
10. Was the building a new job or a remodeled job?

If the walls are of brick answer the following questions:

11a. Were the walls new or old when stuccoed?
11b. Were the bricks new or old?
11c. Were the joints raked out?
11d. Were the bricks painted?
11e. Were the bricks hacked?
11f. Was wire lath or metal lath used to bond stucco to wall? (State kind if known.)
11g. Was wire lath or metal lath used in the ornamental work, or for other purposes?
11h. Was furring used?
11i. What kind of furring?
11j. Was tar or asphalt waterproofing of any kind used on the walls?

If the walls are of terra cotta tile, answer the following questions:

12a. Was the tile scored?
12b. Was metal lath or wire mesh used over the tile? (State kind if known.)
12c. Was tar or asphalt waterproofing of any kind used on walls?

If the walls are of reinforced concrete answer the following questions:

13a. Was the surface treated or not before the stucco was applied?
13b. Was the surface wire brushed, tooled, grouted, coated with tar or asphalt waterproofing before the stucco was applied?
13c. Was the surface stuccoed green as soon as forms were removed, or after drying out?

If the walls are of concrete block answer the following questions:

14a. What size were the blocks?
14b. Were the blocks rough or smooth?
14c. Were the blocks treated in any way?

If the walls are of frame, answer the following questions:

15a. Was the sheathing or boarding horizontal or diagonal?
15b. Was the sheathing or boarding covered with paper?
15c. What kind of paper?
15d. Was metal lath nailed direct to studs, no sheathing being used?
15e. Was the lath back plastered?
15f. Was the lath furred from sheathing or studs?
15g. What kind of furring was used?
15h. What kind of lath was used? (State whether galvanized, painted, or uncoated, and the make if known.)
15i. Was wood lath used?
15j. What kind of wood lath?
15k. Was it plain lathed or cross lathed?
15l. Was furring used, and if so what kind?
15m. Was the lath coated with tar, asphalt or creosote?
15n. What size keys were left?
15o. Was patent lath of any kind used?
15p. If so, what make?
15q. If this was an old building "overcoated," state kind of lath used and how it was applied.

16. Would you call this job a particularly good, a good, a fair, a poor one?
17. Were the mechanics who did the work first class, fair, or poor?
18. Was the job rushed?
19. Was it done as well as possible?
20. Is the job plain, with no ornament?
21. Is it of special artistic design?
22. Is the color ordinary or special?
23. If special what color is it?
24. Is the finish sand float, rough cast, splatter dash, pebble dash, or special finish?
25. If special finish, what kind is it?
26. Is the job ornamental?
27. Did they run the cornices, belt courses, base courses, sills, parapet copings, window and door trim?
28. Did they cast the cornices, belt courses, base courses, sills, parapet copings, window and door trim?
29. Was the work run with regular sheet iron molds horsed up on wood?
30. Was the work run with wood molds, no sheet iron being used?
31. Was the ornamental work and trim cast in glue, wood or plaster molds?
32. Is the job protected from the weather by an overhanging roof, or by flashing on top of projecting stucco moldings?
33. If there is a parapet and coping in stucco are they flashed over the top?

34. Are the cornices, copings, moldings and trim made of galvanized iron?
35. Are the cornices, copings, moldings and trim made of wood?
36. Has the stucco been damaged by leaks or drip?
37. Has the architect made special provision to protect the work from leaks and drip?

38. Was river sand or bank sand used in the stucco?
39. Was the sand fine or coarse?
40. Were fine or coarse pebbles used?
41. Was grey or white portland cement or both used?
42. Was natural cement used? (Such as LaFarge, Rosendale, etc.)
43. Was magnesia cement used? (Such as Kellastone and similar products bearing trade names.)
44. Was slaked lump lime, or hydrated lime used?
45. Was any other kind of plaster or cement used?
46. Was hair or fiber used in the scratch coat?
47. What mix was used in the first coat?
48. What mix was used in the second coat?
49. What mix was used in the finish coat?
50. Was the first coat scratched and allowed to dry before the second coat was put on?
51. Was the second coat scratched and allowed to dry before the finish was put on?
52. Were the first and second coats laid on together then scratched and allowed to dry before the finish was put on?
53. Was the first coat scratched and allowed to dry before the second and finish coats were put on, (these last tow coats being put on as one)?
54. Was the stucco put on in any way different from those indicated above? If so, state how.

55. Do you see fine cracks on the stucco which are known as craze cracks, map cracks, fire cracks, hair cracks, (all meaning the same thing)?
56. Are there many of them or few?
57. Can you see them from the street, say 25 ft. away?
58. Are the most of them on the north, south, east or west side?
59. Are there many or few large and prominent cracks?
60. Are they at the corners of the windows and doors?
61. Are they running up and down or across the wall?
62. Are there any loose places?
63. Are there any broken places?
64. Has the stucco fallen off?
65. Can you tell whether the metal lath has rusted or not?
66. Is the color even or uneven?
67. Is the texture even or uneven?

68. Are there any scaffold laps?
69. Do you think the cracks are due to shrinkage of the stucco, movement of the brick or tile walls, warping of the frame walls, laps in the metal laths, or buckles in the wood lath?
70. What do you think caused the cracks?
71. Are the chimneys as good or not as good as the walls?
72. Are the sills in good, fair or poor shape?
73. Is the ornamental work in good, fair or poor condition?
74. Is the ornamental work in as good or not as good condition as the plain stucco?
75. Was the same mix used in the ornamental as the plain work?
76. What mix was used in the ornamental work?
77. Was dry cement dusted on the moldings to help finish them quickly?
78. Was the stucco carried to the ground?
79. Is it injured by doing so?

80. Have repairs been made?
81. Have the repairs been well, fairly or poorly made?
82. Do you think the stucco will fall off?
83. Do you think the stucco will last a long time?
84. Has the stucco been painted?
85. What kind of paint was used?
86. Is the paint in good, fair or poor condition?
87. Is it peeling off?
88. Is it blotchy and changing color?
89. Do you like the appearance of the job?
90. What do you think could have been done to improve the job?
91. What do you think could be done to improve it now?
92. What kind of waterproofing was used, if any?
93. Are there any other interesting points about this job not covered by the foregoing questions?

Inspected, date..
By..
Address..

The committee desires to call attention to the importance of obtaining a large number of these records from all parts of the country in order that not only all types of buildings may be represented but also a wide variety of climatic conditions may be covered.

With reference to concrete surface treatments, the committee plans to obtain records of existing structures in a manner very similar to that described above for stucco. The information blanks are patterned quite closely after those relating to stucco and contain the following headings:

1. Type of structure.
2. Owner.

3. Location.
4. Architect.
5. Builder.
6. Estimated or approximate cost of structure.
7. Date of erection.
8. Surface treatment. (Give brief description of color, texture, mechanical or other treatment, mixtures, type and proportions of aggregates, etc.)
9. Descriptive articles.
10. Photographs.
11. Remarks. (Describe briefly present condition, prominent defects, if any, effectiveness of surface treatment, etc.)

Submitted by.................. Date..............
Address..

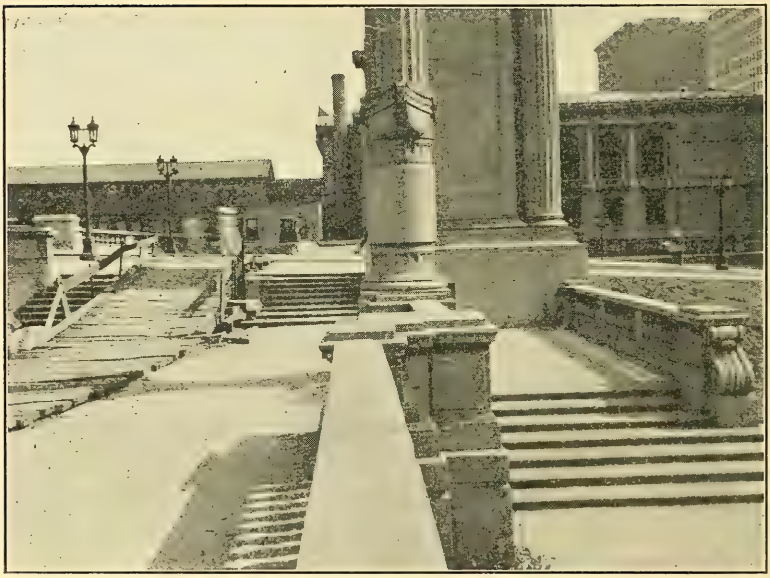


FIG. 1.—DISPLACEMENT OF CONCRETE BALUSTRADE PEDESTAL CAUSED BY EXPANSION OF THE LONG HAND RAILS.

The photograph shows this displacement chiefly in the divergence of the lines of the railing and pedestal cap.

Assuming that the chief interest lies in those surface treatments which are in some degree special, the structures with which we are concerned are more or less of a monumental type and therefore comparatively few in number (that is, as compared with the number of stucco structures) and many of them, perhaps the majority, have been the creations of individuals or small groups.

of individuals working up their own specifications to meet their own particular requirements without any well established technique to serve as a guide in securing the results desired.

This lack of established technique at the present time indicates slow development in the art of concrete surface treatment, and the impression is gained that this slow development results partly from mediocre success in ambitious attempts, partly from the high cost of elaborate treatments, but mainly from a lack of knowledge of the peculiarities and possibilities of the material itself.

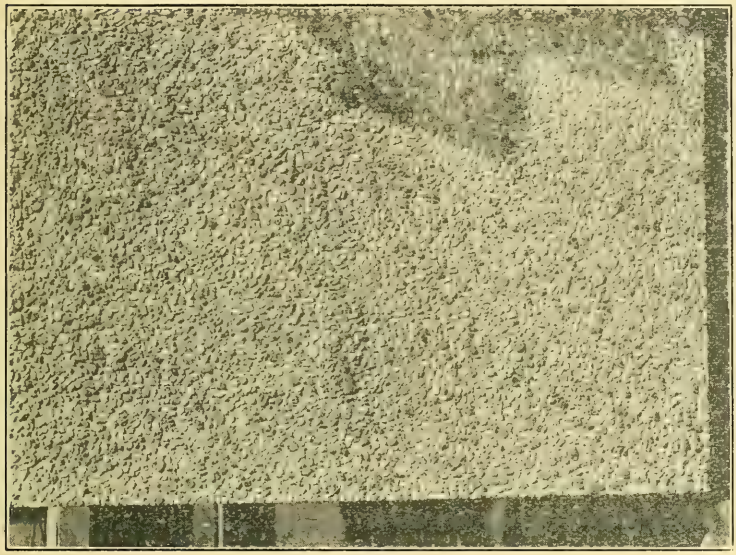


FIG. 2.—EXAMPLE OF CONCRETE SURFACE TREATMENT SHOWING CLOSELY PACKED ROUNDED AGGREGATES.

Observed from a moderate distance the color is buff, pleasingly varied by the presence of occasional darker aggregates.

For example one of the peculiarities of concrete that cannot be disregarded is volume change under changing moisture conditions. It is of course well known that concrete expands when wet and contracts when drying out, but hardly enough importance seems to have been attached to this property in the design of many ornamental structures. This fact is illustrated in Fig. 1, which shows a portion of the southern end of the long balustrade in Grant Park, Chicago, which was described in the "Proceedings" of this Institute for 1917. The photograph was taken with a small pocket camera from directly above the center line of the balustrade looking south.

It is quite readily seen in Fig. 1 that the cumulative expansion of the long railing has exerted sufficient force against the heavy post to turn the latter appreciably on its vertical axis. This movement has of course resulted from the fact that the two sections of balustrade on either side of the post are parallel, but not in line. The resulting displacement would scarcely have been anticipated, and yet might have been prevented by careful design and proper joint treatment, which is a real problem in structures of this type. Numerous other evidences of the movement of the balustrade may be found on careful inspection, the chief of which are spalling of the edges of the railing at several of the joints. Insignificant as the defects may appear to the casual observer, they call attention to the fact that the fundamental properties of concrete must not be overlooked in a work of this sort.

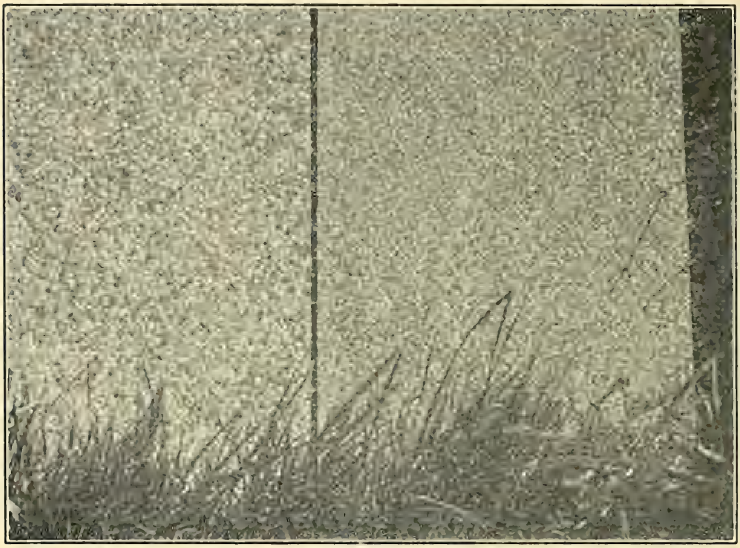


FIG. 3.—CONCRETE SURFACE TREATMENTS SHOWING AGGREGATES OF CRUSHED RIVER GRAVEL, MAINLY QUARTZ.

The specimen on the right shows a uniform fine texture and delicate buff color, the one on the left a coarser texture and a warmer tone produced by frequent spots of red.

Figs. 2 and 3 are close views of some samples prepared and submitted by one of the members of this committee for certain Government work in Washington. They are shown merely to illustrate that real progress is being made in the improvement of colors and textures by suitable concrete surface treatments.

Fig. 2 represents a texture produced by the use of Potomac River Gravel from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in size as the large and predominating aggregate in a carefully proportioned mixture. The pebbles were exposed by wire brushing and acid washing.

Fig. 3 represents textures obtained mainly by using crushed Potomac River gravel as the coarse aggregate, the slab on the right being of $\frac{1}{8}$ in. to $\frac{1}{4}$ in. material, the one on the left of $\frac{1}{4}$ in. to $\frac{1}{2}$ in. material dotted here and there with spots of red brick. Unfortunately the colors do not reproduce, but there is a warmth in these surfaces entirely foreign to ordinary concrete, due to the fact that the colors are almost wholly governed by those of the aggregates. The most important advance, therefore, consists in the fact that in such material the architect has at his disposal a medium for the expression of widely varying colors and textures, obtainable in casts of any required form.

The illustrations accompanying this report are particularly suggestive of the peculiarities and the possibilities of ornamental or architectural concrete. Success in surface treatment of concrete requires first of all that its peculiarities and limitations be carefully recognized, and second, that whatever beauty it may attain is vastly more than skin deep. It appears, therefore, that to develop successful and pleasing treatments of character and variety, the most fundamental theories of concrete structures must be given attention, and the results of such study must be combined with high technical skill. The committee believes that both stucco and concrete surface treatment are capable of being greatly improved and developed, but the latter in particular seems to offer most interesting possibilities. It is therefore to discover and report upon the present status of the art, as a basis for further development, that the committee has decided to gather together the data on existing concrete structures which have received special surface treatment.

Members of the Institute are cordially invited to furnish the committee records of stucco or concrete structures of which they have knowledge, and suggestions as to the sources of similar information will be gratefully received.

Respectfully submitted,

E. D. BOYER,
W. H. BOUGHTON,
L. R. FERGUSON,
Z. W. CARTER,
C. M. CHAPMAN,
J. B. ORR,
J. J. EARLEY,
J. E. FREEMAN,
J. C. PEARSON, *Chairman.*

REPORT OF COMMITTEE ON BUILDING BLOCKS AND CEMENT PRODUCTS.

During the past twelve months it has been impossible to get the members of this committee together to do real work on investigations of concrete products, for the purpose of submitting additional specifications to the institute. Only three members of this committee have found it possible to attend meetings. On account of the shortage of technical men, due to the participation of our country in the world war, every engineer is greatly pressed for time and is required to handle so much more work than heretofore that little time can be spared for committee work. So, after careful consideration, our committee decided to limit its investigations to but one product; that of concrete roofing tile.

This is made in large units about 2 ft. wide and 4 ft. long and in small sizes about 12 x 14 in., similar to the clay roofing tile. We decided to limit our investigation to the small size.

In order to make an intelligent investigation it was necessary to secure samples from as many manufacturers as possible. Our secretary, Mr. Meriwether, was unceasing in his effort to secure these samples and after five months' correspondence with different manufacturers, succeeded in getting 64 samples from seven different manufacturers. Prof. D. A. Abrams, of the Structural Materials Research Laboratory of Lewis Institute, Chicago, very kindly consented to make the tests. These samples were not received early enough to allow test to be made in sufficient time to enable us to get up a specification covering the use of the tile.

We found that there is practically but one type of small concrete roofing tile made. We, therefore, decided to attach the report of the tests to this report without comment, leaving the drawing up of the specification to the next committee.

The chairman of this committee is greatly indebted to Prof. D. A. Abrams and Mr. Meriwether for the time and effort they have devoted to this work.

Robert F. Havlik, *Chairman,*
Coleman Meriwether, *Secretary,*
D. A. Abrams,
John K. Harridge.

Tests of Concrete Roofing Tile.

(To accompany report of Committee on Building Blocks and Concrete Products.)

The purpose of the tests was to secure a basis for specification requirements for concrete products of this kind.

The tile were shipped to the laboratory by the manufacturers at the request of the committee.

All tile were of the "Walter" type. The name of the manufacturers, and all other information that was furnished regarding the mix, age, etc., is given in Table 1.

TABLE 1.—DATA OF CONCRETE ROOFING TILE.

Ref. No.	From Whom Received.	Mix.	Aggregate.	Color.
1	L. Hansen, Kansas City, Mo...........	1:2	Kaw River Sand	
2		1:2	" " "	
3		1:2	" " "	
4		1:2	" " "	
5		1:2	" " "	
6		1:2	" " "	
7		1:2	" " "	
8		1:2	" " "	
9		1:2	" " "	
10		1:2	" " "	
11*a*		1:2*a*	" " "	
12*b*		1:2*b*	" " "	
13*c*		1:2*c*	" " "	
14		1:2	" " "	
15	Walter Concrete Machinery Co., Indianapolis, Ind.	...	....	Red
16		...	....	"
17		...	....	"
18		...	....	"
19		...	....	"
20		...	....	"
21		...	....	"
22		...	....	Gray
23		...	....	"
24		...	....	"
25		...	....	"
26		...	....	"
27	Hawthorne Cement Products Co., Chicago, Ill.	1:3	Garden City Sand	Red
28		1:3	" " "	"
29		1:3	" " "	"
30		1:3	" " "	"
31		1:3	" " "	"
32		1:3	" " "	"
33	Universal Roofing Tile Co., Chicago, Ill.	1:3	....	Birdsal
34		1:3	....	Red
35	Empire Tile Co., Cleveland, Ohio.......	1:3	....	Painted Red
36		1:3	....	" Green
37		1:3	....	" Gray
38		1:3	....	" Red
39		1:3	....	" Slate
40		1:3	....	" Green
41	P. & R. Concrete Roofing Tile Co., Buffalo, N. Y.	1:3	River Grit and Bank Sand	Red
42		1:3	" " " " "	"
43		1:3	" " " " "	"
44		1:3	" " " " "	"
45		1:3	" " " " "	White
46		1:3	" " " " "	"
47		1:3	" " " " "	"
48		1:3	" " " " "	"
49		1:3	" " " " "	"
50		1:3	" " " " "	Red
51		1:3	" " " " "	"
52		1:3	" " " " "	"
53	Hawthorne Cement Products Co., Chicago, Ill.	1:3	Garden City Sand	Dark Red
54		1:3	" " "	" "
55		1:3	" " "	" "
56		1:3	" " "	" "
57		1:3	" " "	Light Red
58		1:3	" " "	" "
59		1:3	" " "	" "
60		1:3	" " "	" "
61*d*		1:3	" " "	" "
62*d*		1:3	" " "	" "
63*d*		1:3	" " "	" "
64*d*		1:3	" " "	" "

a One-fourth part dust of crushed concrete street paving.
b One-half siliconite.
c Burned clay used in mixture.
d Appeared very porous.

TABLE 2.—DATA OF TESTS OF CONCRETE ROOFING TILE.

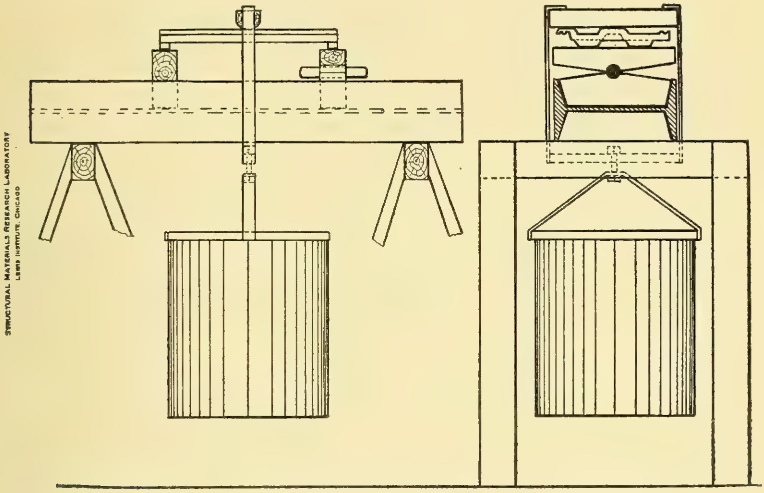
Ref. No.	Weight of Tile, lb.	Width of Tile, in.	Span Length, in.	Thickness of Tile, in.	Age at Test, mos.	Center Breaking Load, lb.	Absorption, per cent weight.
1	4.57	8.9	14.3	0.44	23	151	8.8
2	4.42	8.9	14.3	0.42	9	92	9.1
3	4.65	8.9	14.3	0.45	9	128	8.7
4*	....	...	....	....	..	...	9.4
5	4.18	8.8	14.3	0.40	9	99	9.6
6	4.65	8.9	14.3	0.45	9	120	9.7
7	4.65	8.8	14.3	0.43	9	103	9.7
8	4.45	8.8	14.3	0.42	9	119	9.4
9	4.50	8.8	14.3	0.44	21	158	8.3
10	4.34	8.9	14.3	0.40	21	145	9.4
11	4.52	8.8	14.2	0.45	7	88	10.9
12*	....	...	....	....	..	...	7.5
13*	....	...	....	....	..	...	8.1
14*	....	...	....	....	..	...	12.0
15	5.96	9.2	13.8	0.47	8	147	10.1
16	5.37	9.2	13.8	0.42	8	230	10.3
17	5.96	9.2	13.8	0.48	8	251	11.0
18	5.84	9.3	13.8	0.45	8	207	10.6
19	5.95	9.2	13.8	0.46	8	256	10.1
20	6.05	9.3	13.8	0.49	8	250	10.5
21	5.64	9.2	13.8	0.45	8	235	10.6
22	5.84	9.2	13.8	0.47	8	216	10.2
23	5.97	9.2	13.8	0.50	8	255	11.3
24	5.84	9.3	13.8	0.46	8	119	10.2
25	5.50	9.2	13.8	0.44	8	217	10.5
26	5.95	9.2	13.8	0.46	8	260	10.1
27	5.28	9.2	13.8	0.44	..	81	12.5
28	5.30	9.2	13.8	0.46	..	95	12.3
29	5.97	9.2	13.8	0.50	..	114	11.1
30	5.60	9.2	13.8	0.49	..	128	15.3
31	6.15	9.3	13.8	0.49	..	165	12.5
32	5.30	9.2	13.8	0.50	..	65	16.2
33	5.60	9.2	13.8	0.47	..	100	15.2
34	5.56	9.2	13.8	0.45	..	137	13.4
35	5.77	9.2	13.8	0.45	8	206	11.3
36*	....	...	....	....	..	...	....
37*	....	...	....	....	..	...	....
38	5.50	9.2	13.8	0.43	8	102	10.6
39	6.38	9.2	12.9	0.49	8	109	10.9
40	5.68	9.2	13.8	0.46	8	207	11.1
41	5.38	9.2	13.8	0.43	5	89	14.0
42	5.56	9.2	13.8	0.45	5	102	14.4
43	5.48	9.2	13.8	0.45	5	89	14.8
44	5.56	9.2	13.8	0.45	5	84	13.5
45	5.60	9.2	13.8	0.43	5	118	14.0
46	5.22	9.2	13.8	0.43	5	148	14.2
47	5.40	9.2	13.8	0.46	5	92	14.6
48	5.37	9.2	13.8	0.45	5	143	13.6
49	5.17	9.2	13.8	0.42	5	149	11.6
50*	....	...	....	....	..	...	13.6
51*	....	...	....	....	..	...	13.7
52*	....	...	....	....	..	...	12.3
53	5.78	9.2	13.8	0.47	22	190	9.9
54	5.80	9.2	13.8	0.46	22	178	10.0
55	5.88	9.2	13.8	0.49	22	201	10.5
56*	....	...	....	....	..	...	9.8
57	5.72	8.2	13.8	0.49	14	110	13.0
58	6.12	9.2	13.8	0.52	14	156	12.6
59	5.61	9.2	13.8	0.45	14	194	11.4
60*	....	...	....	....	..	...	11.6
61	5.45	9.2	13.8	0.51	14	65	15.0
62	5.50	9.2	13.8	0.50	14	114	15.6
63*	....	...	....	....	..	...	16.7
64*	....	...	....	....	..	...	16.0

* Broken in transit.

The tests consisted in determining the weight, center breaking load and absorption.

The home-made device for loading the tile is shown on the accompanying sketch. The low loads made the use of an ordinary testing machine undesirable. The tile was supported on the four end lugs with the weather face up. The span length was measured between centers of lugs. Load was applied by placing metal weights in the bucket, then gradually admitting sand until failure occurred. The weight causing failure was determined by platform scales.

The tile was mounted for test in such a way that no twisting was developed, although all of the lugs may not have been in the same plane. The accompanying drawing shows how this was accomplished by means of a rocker support at one end.



APPARATUS FOR TESTING CONCRETE ROOFING TILE.

Practically all tile failed directly under the center loading block.

The thickness recorded in Table 2 is the average of three or four measurements at the section of failure.

On account of the irregular section of the tile no attempt was made to compute the modulus of rupture of the concrete.

Absorption tests were made as follows:

A piece from near the middle of each tile, about 12 to 20 sq. in. in area was dried to constant weight; placed in water; water heated gradually, then boiled for 5 hrs. tile allowed to cool in water; surface water removed; test specimen weighed. The gain in weight expressed as a percentage of the original dry weight is the absorption.

D. A. ABRAMS,
Professor in Charge of Laboratory, Structural Materials
Research Laboratory, Lewis Institute, Chicago.

DISCUSSION.

Mr. Chapman. Mr. Cloyd M. Chapman.—I am pleased to note that the committee is taking up the subject of tile. We have had considerable trouble with tile because of brittleness. They will not stand handling. We find a number of them broken in the middle after they are laid, particularly if they are not laid perfectly. Another trouble is warping on the roof.

On the other hand, we find a great many tile that are extremely good. We have made some tests in the laboratory on tile of a larger type, 18 in. by 3, 4 and 5 ft. These tests have been made for brittleness. Some of the tile would not stand up when a block of wood was dropped on them from a height of 6 ft. In some cases the block would cause the tile to ring like a bell and would rebound well into the air. Such tile were very tough and durable but others were so brittle that the men would break them in walking on the roof. If the committee will bring the low grade of tile up to the high grade it will have done a great work.

Mr. Humphrey. Mr. Richard L. Humphrey.—I would like to ask Mr. Chapman what ought to be the elements of a specification for concrete roofing tile.

Mr. Chapman. Mr. Cloyd M. Chapman.—Our greatest trouble was lack of toughness; breakage of the tile on the roof after it was laid. I do not know about the difficulties of allowing the tile to stand out in winter weather. As I said we test the tile by dropping on it a block of wood but we have also used a bag of sand. Very often this would be dropped at a height of 3 ft. and would break the tile. The wood block is just as satisfactory as the sand.

Mr. Schouler. Mr. W. H. Schouler.—The tile described in the committee's tests is controlled by patents. They are made by a semi-wet process and the trowel is shaped to give the corrugations on the face of the tile. The manipulations of the trowel bring the mixture to the surface and tends to make it water-tight and non-absorbent.

REPORT OF THE COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

At the meeting of the American Concrete Institute in Chicago in February, 1917, a proposed Standard Building Regulation for the Use of Reinforced-Concrete* was submitted for adoption. Later in the convention it was ordered that the report go over for another year without action.

During the past year some very valuable data have been obtained from several tests, including:

(*a*) Tests of two American Can Company buildings, Maywood, Ill.

(*b*) Tests of an eight-year-old flat slab floor of the Western Newspaper Union Building.†

(*c*) Tests of four flat slabs at Purdue University, by Professor W. K. Hatt.†

Your committee wishes to study these tests before bringing the proposed Standard Building Regulations before the Institute again for adoption.

Of this committee, three members are in the service of the U. S. Army, and the other members have been employed upon war work to such an extent that it has been impracticable to have committee meetings during the past year.

It is suggested to the convention that the committee be allowed additional time until war conditions permit the committee having meetings and until the new test data can be studied by the committee.

E. J. Moore, *Chairman.*

Capt. A. R. Lord, of the committee, in his approval of the report suggests the following addition:

"New data secured by the Structural Research Section of the Concrete Ship Department of the Emergency Fleet Corporation appear to call for considerable revision of regulations governing shear in beams and some additions to other sections of our report."

This has not been incorporated on acount of there not being time to bring it to the attention of the other members of the committee.

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* Printed in *Proceedings*, American Concrete Institute, Vol. XIII, 1917, p. 410.

† Reported on p. 192 and 164 of this volume.

REPORT OF THE COMMITTEE ON INDUSTRIAL CONCRETE HOUSES.

Owing to the late date at which the committee was appointed, the work done must be considered as simply of a preliminary nature. Some information has been gathered and some exchange of views has been made between the members of the committee in the report as now put forward and the papers already presented at the convention, some idea will be given of the scope of the subject and its importance to the country in general and to the American Concrete Institute in particular.

The concrete house is now an accomplished fact. It has come to stay. For many years experiments have been made with many types. Many of the earlier houses were quite unsuitable and showed defects, but the later examples are much improved, the earlier defects are being eliminated and the concrete house as built today is a thoroughly sanitary, weatherproof, permanent and fireproof home. It approaches more nearly the ideal home than any other type of construction.

DEFINITIONS.

The expression, "concrete house" is a rather indefinite term conveying different meanings to different minds. In order to make this report clear the committee have defined the concrete house as a house having external walls of monolithic concrete precast concrete or concrete blocks. A house which also has internal partitions and floors and roof of concrete we describe as "all-concrete house." A frame building covered with stucco or gunite is not considered to fall within the definition "concrete house" and is defined as a "cement covered house."

CONDENSATION TO BE GUARDED AGAINST.

We have found that there exists in the minds of the layman an impression that concrete is damp and unhealthy. It is most unfortunate that concrete should have such a reputation even though we have found some earlier houses that give some justification for this criticism. It will be recognized by members of the Institute that while a concrete wall may be perfectly waterproof it is subject to condensation on a wet day unless the walls are furred and plastered so as to get an air space insulation; this condensation is sure to be apparent. In some houses this has proved to be a very objectionable feature; in other houses inspected by members of our committee the tenants complain that on the family wash day the walls were very wet and on other days they were comfortable. Climate has a good deal to do with this.

RELATIVE COSTS.

The investigation of the committee have not shown that a concrete house can be built more cheaply than a wooden frame house. The relative cost of course varies in different localities according to the availability of different types of material, but speaking generally we find that the cost of a concrete house should run from 10 to 15 per cent more than the cost of a well constructed frame house.

The advocates of the concrete house must rely upon the superior merits of concrete as a material and not upon its low first costs.

THREE TYPES OF CONSTRUCTION.

Three principal types of concrete house construction are the concrete block, the precast house and the monolithic house. We have not pushed our studies into the stucco house or cement-gun covered house, which seem to be outside of the scope of the committee. The general conclusions of the committee as to the merits of the various methods are that for small jobs the concrete block house stuccoed on the outside is the best solution but for large developments in which more than fifty or sixty houses are required the monolithic or precast types, show an economy in cost and speed that should place them in the foreground.

DIFFICULTIES TO BE OVERCOME.

In any type of concrete house construction difficulties are sure to be met with. Some of these are discouraging to the contractor entering this field and are likely to hinder its expansion unless faced and overcome. The committee have thought it well to discuss them frankly in order to guard the novice against disappointment and failure. Realizing as we do that these difficulties are incidental only and not insuperable. The successful work done is sufficient answer to any objection raised that concrete house construction is too difficult or impossible.

PRECAST CONSTRUCTION.

The precast house composed, as it is, of shop made units of considerable size and weight needs a large outlay on equipment for hoisting erection and transportation of the units. This limits the use of this method to groups of houses which are large enough to amortize this outlay in a reasonable time. Great care also is needed in setting to prevent breakage—as a broken unit not only involves loss of spoiled material but considerable delay while fresh units are made. In order for a precast system to show economy the design of the houses must be alike in plan and uniform in design.

MONOLITHIC CONSTRUCTION.

The committee are agreed that wood forms are out of the question for monolithic house construction. Although the first cost is low the life is short, the upkeep high, and the results obtained are generally unsatisfactory. In any kind of concrete construction the concrete wall is more difficult to form than the floor, the foundations or other parts of the building, and it is therefore necessary to discard the wood form and search for something better if monolithic concrete houses are to be built successfully.

There are about a dozen metal wall forms for house construction on the market. One of these is made of structural steel channels—the rest of sheet metal. With certain forms a complete story can be poured at one time but the greater number are designed to allow the pouring of one course (usually 2 feet) at a time.

The structural steel form has the demerit of being exceedingly heavy but on the other hand is easily set up true to line and giving a very good wall surface. The sheet metal form is not so easily kept true and is easily distorted and damaged but with proper care can be used to produce excellent results.

EXTENT OF APPLICATION OF CONCRETE.

The committee has not come to any conclusion as to how much of the house should be built of concrete. Accordingly the members have presented their divergent views in their papers rather than attempt to come to any agreement so quickly. It is certain that the concrete wall offers great advantages due to its fireproof, weatherproof and permanent qualities. It should be furred inside. It should cost about the same as a brick wall or slightly less. A concrete floor should cost about 18 c. per sq. ft. more than a wood floor. Concrete with wood top surface about 30 c. per sq. ft. more.

A granolithic floor is advocated by some on account of the ease of cleaning, but for poor families who cannot afford many rugs it is unsuitable and it is unhealthy for small children to play on a cement floor.

When concrete construction is used for constructing outside features like porch columns and roofs and moldings, cornices and ornaments the cost of the house is increased very quickly.

EXTERNAL SURFACES.

Concrete being a new material, the tendency is generally to cover it up to make it look like something else, and therefore most of the concrete houses built up to the present time have been covered with stucco. With a steel form that gives a good even line concrete can be finished with a rubbed, or wire brushed or tooled surface that will be quite satisfactory in texture and in color. And very artistic effects can be gained with the use of colored aggregates. To finish with a wood float does not give a good result. The concrete house is superior to brick or any other type of house on account of its permanency, fireproof and weatherproof qualities and pleasing appearance. No repointing is necessary, no efflorescence disfigures its surfaces. The all concrete house combines with these advantages the absolute rigidity which is lacking in a wood floored house, sticking of doors, settlement cracks in walls and in plastering are avoided and there is no harborage for vermin.

RESTRICTION LAWS.

Building ordinances framed to control brick tile and stone construction before the extensive use of concrete in wall construction are in most cases unduly burdensome and restrictive when applied to concrete. Six-inch walls are the maximum needed in monolithic construction and 2 in. is common in precast construction where other structural members carry the load. Most codes call for 8 and some as much as 12-in. thickness. Similar difficulties are found with floors. A reform of our building laws is urgently needed in many of our big cities and the committee has in mind the preparation of a building code for concrete small house construction which will serve as a model for cities and suggest, that after receiving the endorsement of the Institute it be circulated and urged upon municipal authorities.

There is a widespread interest in the concrete house today. There is not, however, much active demand for them, owing to the dearth of contractors experienced in concrete work who are entering this new field to create a supply. The majority of the small houses built today are built for quick sale by the vendor and not for investment. House building standards have been low. Competition in prices has been keen but quality has been a secondary consideration. The demand at the present time seems to be for a better type of construction—concrete meets this demand.

VALUE OF PERMANENT CONSTRUCTION.

The housing program of the Government entrusted to the Shipping Board and the Department of Labor will set the standard of housing for many years to come. It is of the utmost importance that these standards should be higher than the low standards that now prevail and especially that the work done be of such permanent character that it may prove to be an adequate security for the funds expended and for long term mortgage bonds at low rates of interest.

We believe that the members of the Institute should use their influence to ensure that in the carrying out of this program and in the other industrial housing work carried out by Public funds or private enterprise the concrete house should occupy its rightful place.

Good and sufficient housing is one of the most pressing needs of the nation at this time. Without it we are seriously handicapped in the winning of the war. The provision of good housing is not only an immediate need but a permanent national gain and that every member who lends his influence and aid to the securing of this end is rendering a real service to the nation.

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Secretary.

REPORT OF COMMITTEE ON SIDEWALKS AND FLOORS.

Since the last report of the committee specifications for sidewalk construction have been secured from over a hundred cities throughout the country, digest of which has been prepared and made a part of this report. Since the question of subbase for sidewalks merited special consideration, the accompanying letter was sent to city engineers in those cities shown by the previous survey to have constructed walks on the natural ground.

Replies received have been abstracted as shown on the accompanying folded table. After a study of this subject the recommendation seems warranted that the subbase of cinders, gravel or similar material be eliminated wherever possible and the walk laid directly on the properly prepared subgrade. Proper preparation will include provision for adequate drainage in heavy soils of claying character.

Success in the use of the roller for finishing concrete pavements, also the canvas belt, suggests their desirability for sidewalk construction, especially one-course walks, and the committee believes that those members concerned with such work should give this due consideration. The canvas belt should produce a surface of superior texture and quality and give a better foothold than steel troweled work. Several walks are planned to be built this season with roller finish and results will be reported later.

The wear tests of concrete for road construction that have been made at the Structural Materials Research Laboratory indicate that this would be a suitable method for measuring the wearing qualities of different methods and mixtures for floor construction, and with the cooperation of the laboratory, a series of tests is being planned which it is expected will develop very valuable and interesting data on the effect of grading or character of aggregate, proportions, consistency, method of finishing, protection while hardening, etc., upon the wearing qualities of concrete floor surfaces. Such data will undoubtedly go far toward developing the best methods of construction that will produce a floor with the high resistance to abrasion so necessary in industrial buildings.

The accompanying specifications for floors and for sidewalks are submitted as proposed revised standards of the Institute

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P. M. Bruner,
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L. R. Ferguson,
E. G. Perrot,
W. N. Rynerson,
J. E. Freeman, *Chairman.*

COPY OF LETTER SENT TO CITIES.

The Committee on Sidewalks and Floors of the American Concrete Institute is at this time especially interested in the matter of concrete sidewalks constructed without the use of a subbase, either of cinders, gravel, crushed stone or similar materials.

On looking over the sidewalk specifications for your city, I notice that no subbase is specified, or if so, only in particular cases. You, therefore, are undoubtedly in a position to furnish valuable information to this committee which will be of considerable assistance in this investigation. Such data as you can give us on the reasons for the omission of the subbase, how the drainage is taken care of, the results obtained, the cost of walks and your personal opinion on the subject will be greatly appreciated.

Yours truly,

Chairman, Committee on Sidewalks and Floors,
American Concrete Institute.

ABSTRACTS OF LETTERS FROM CITY ENGINEERS, ETC., ON CONSTRUCTION OF CONCRETE WALKS WITHOUT SUBBASE.

Boise, Idaho.—No subbase required because of good natural drainage.

Sioux City, Iowa.—Use no other foundation than natural soil, and take special care to remove the excess water in concrete when finishing.

Portland, Ore.—No walks have been laid other than on natural soil. Climate comparatively mild. A few cases of frost heaving observed, but walks settled down to original grade.

Birmingham, Ala.—Practice to lay walks directly on subsoil, usually clay, and, while range of temperature is from over 100 to below zero, no trouble experienced.

New Orleans, La.—Do not find subbase necessary. Subgrade and walk surface sloped so that surface water flows toward curbing and subsurface drains.

Grand Forks, N. D.—Clay soil overlaid with 2 ft. black earth. Frost reaches 5-ft. depth. Abandoned filling under walks in 1908. Expense less and results as good if not more satisfactory.

Salem, Ore—Not troubled by frost. Walks laid on natural ground. Under side of slab 6 to 10 in. above gutter line of street, which provides sufficient drainage in this locality.

Los Angeles, Cal—Two-inch sand cushion used for walks laid in adobe soil, not to relieve frost action, but to prevent trouble from swelling of soil during rainy season.

Cleveland, Ohio.—Soil largely sandy, affording good natural drainage, but good results also from walks laid directly on clay soil. Have not yet decided to omit subbase altogether, but believe in most cases subbase is an additional expense with no added security.

South Bend, Ind.—Sandy subsoil. Walks laid on natural surface give no trouble.

Memphis, Tenn.—Little trouble from freezing. Original specifications provided for 6-in. subbase but not for subdrainage. Considered waste of money and later changed to lay walks directly on clay foundation. Believes construction tends to cause water to flow over surface rather than down underneath.

Baltimore, Md.—Specifications were changed in 1914 to omit subbase, and so far results have been satisfactory besides cutting cost about one-third.

St. Joseph, Mo.—Construction of walks without subbase begun three years ago now made general practice. Subbase considered unnecessary and work done more cheaply. Soil mostly sandy clay. Tile drains used in some areas, also cinders where soil naturally holds large percentage of water the year around.

Moline, Ill.—In 1914 specifications changed from two-course walk with 8-in. subbase to single course with no subbase. Have laid about 25 miles of

DIGEST OF SPECIFICATIONS FOR CONCRETE SIDEWALK CONSTRUCTION IN AMERICAN CITIES.

PREPARED BY COMMITTEE ON SIDEWALKS AND FLOORS FROM ANSWERS TO CIRCULAR LETTER.

State	City	Year	Minimum Thickness			Proportions		Size of Aggregate		Method of Mix		Measuring	Foundation (Kind)	Drainage	Size of Blocks	Max. Slope per Foot	Finish	Expansion Joints	Protection	Name Plate	Driveway Thickness	
			Sub-base	Base	Top	Base	Top	Fine	Coarse	Base	Top										Base.	Top.
Minneapolis	Duluth		6 in	3½ in	[illegible] in	2 sand 4 crushed trap granite or gravel	2 sand or 2½ granite screenings 1½ sand 1½ sand screenings	Through ¼ in		M [illegible]	No 1	Boxes	Crushed rock slag coke gravel or stone	3 in clay tile	[illegible] sq ft		[illegible]	Every [illegible] ft	From weather moist 7 days	Street, contractor's name, year	5½ in	½ in.
	Minneapolis	1917	1 in	2½ in	¾ in	4 sand	2 sand			No 2	No 1		Sand		—	—			From sun rain and frost		4 in.	1 in
	St. Paul	1917	4 in	3 in	1 in	3 sand 5 broken stone	2 sand		½ to 1½ in	N[illegible]			Broken stone brick cinders		4 ft long		[illegible]	[illegible] in between slabs	From sun rain and frost			
Nebraska	Lincoln		2 in	3½ in	[illegible] in	[illegible] gravel and sand to fill or 3 sand 6 broken stone	1½ sand 2 screenings	Through ¼ in	Through [illegible] in				Sand or cinders			¼ in	[illegible]	[illegible] between slabs 1 in every 50 ft				
North Dakota	Grand Forks	1917		3½ in	¾ in	[illegible] gravel	1½ sand	Bank gravel screenings	Bank graveled	M	M		Direct on subgrade	As directed	4 ft		[illegible] brush marking	[illegible]	From traffic			
South Dakota	Aberdeen	1916	3 in.	3 in	1 in	[illegible] sand 3 gravel or broken stone	2 sand or screenings	[illegible] to No. 50	¼ to 1 in	M [illegible]	M		Broken brick cinders gravel		5 ft		Template float	¼ in every 5 ft or less	2 in dirt etc. cover, moist 8 days			
	Sioux Falls	1909	6 in	4 in		2 sand 3 crushed rock				No 1			Gravel and broken stone	3 in tile as directed	3 to 6 ft		Trowel to smooth surface	Groove between slabs ¼ in every 1[illegible] ft	Wet earth cover 5 days	Name, year every 150 ft.		
Delaware	Wilmington	1909	11 in.	4 in	1 in.	[illegible] sand [illegible] stone dust 5 gravel or crushed stone	[illegible] stone dust						[illegible] stone 1 to 3 in size	As directed	[illegible] 5 ft	¼ in	Trowel and float grooved [illegible]	¼ in sand through base	From weather wet several days			
District of Columbia	Washington	1916		4 in	1 in	[illegible] sand [illegible] gravel	1½ sand	Fine to coarse	Small to medium	M or H No 1	No 1		On compacted subgrade				Wood batten float toothed roller [illegible]	[illegible] sand joint	Wet and protected until set		4 in.	1 in
Florida	Tampa	1916		3 in broken shell or 4 in clinker	1 in	3 [illegible] sand 5 broken stone or crushed stone or [illegible]	2 river sand or granite sand								Not over [illegible] ft		Struck or floated to smooth surface	Through top and foundation at [illegible]				
Georgia	Atlanta		1 in.	3 in	1 in	[illegible] sand 4 stone	1½ sand		¼ to 1½ in				Subgrade [illegible] sand cushion [illegible] contractor		Min. [illegible] ft max. [illegible] ft		Trowel smooth mark as directed				4 in	1 in
	Augusta	1917		3 in	1 in	2 sand [illegible] stone or gravel	1 sand		¼ to 1¼ in	[illegible] 1	No 1				4 ft square		Trowel with float, then steel trowel	4 to 5 ft apart each way	From sun 10 days wet if directed		4½ in	1½ in.
	Macon			3 in.	1 in	3 sand 5 stone or gravel	1½ sand		Through 1½ in				Direct on subgrade				Information by letter. No printed specifications on hand.)					
Maryland	Baltimore	1916		4 in	1 in	2½ sand 5 stone or gravel	1½ sand		Through 1½ in				Direct on subgrade	Cast iron pipe not less than 4 in	5 ft		[illegible] 2 1 dry mix on top trowel	½ in along curb bitumen between slabs sand	Wet canvas boards until set			
North Carolina	Charlotte			4 in	1 in	[illegible] sand 6 crushed stone	¾ sand ¾ stone dust	Through ¼ in	¾ to 2½ in	M or H [illegible]	No 1	Boxes or barrels	Direct on subgrade		5 ft	¼ in ±	Float and trowel score with roller	Every [illegible] ft	Covered and sprinked several days	Street and number of house		
	Wilmington			4 in	1 in	3 sand 6 crushed stone or gravel	1½ sand	Coarse to fine	Through 2 in	M or H [illegible]	H		Direct on subgrade		Max. 6 x 6 ft	¼ in.	Float and trowel smooth	1 in every 100 ft and between walk and curb	From weather moist 3 days			
South Carolina	Columbia			4 in	1 in	[illegible] sand 4 screened gravel	1 sand 1 screened stone		Through 1½ in								Trowel and float	Tar paper every 10 ft				
Virginia	Norfolk	1916		4 in	1 in	2.6 sand 3.3 gravel or broken stone	1.2 grits or sand		½ to 2½ in	M or H	M or H	Barrels	Direct on subgrade		3 x 3 ft	⅛ in [illegible]	Trowel and float toothed roller	Sand between slabs	From weather moist until set			
	Richmond	1914	4 in	3¾ in	1½ in	2 sand 4 broken granite	1½ screened gravel	Dust to ½ in	Through [illegible] in	M or H No 1	No 1	Forms and boxes	Gravel, sand, cinders		As directed		Trowel dust cement float and trowel hot surface	As directed by engineer	Canvas cover in hot weather, sand several days			
	Roanoke		3 in	4 in.	1 in	2 sand [illegible] stone	1 river sand 1 Vermont or North Carolina granite	Through ¼ in	Through 2 in	H No [illegible]	H No 1	Forms and boxes	Slag		As directed		Trowel dust cement float and trowel dust surface	As directed	Canvas cover in hot weather straw or sand			
Alabama	Birmingham			3 in	1 in	2½ sand 5 slag, stone or gravel	2 sand		Through 1¼ in	M or H No [illegible]	No 1		Firm, tamped direct on subgrade		6 ft. square marked 3 ft sq		Float and trowel with steel trowel	½ in every 50 ft., at curb and corners	Covered planks, canvas wet 10 days			
Kentucky	Lexington		8 in	3 in.	1 in	3½ sand 6½ washed gravel or stone	1½ sand		½ to 2 in	H No 2			Gravel or cinders		6 ft. square	⅜ in	Trowel and float to smooth surface	¼ in every 6 ft and ½ in at curb	From weather, moist until set		5 in	2 in
	Louisville	1917	4 in	4 in.	1 or ½ in.	3 sand [illegible] gravel or crushed stone	2 sand	Through ¼ in	¼ to 1½ in.	M No [illegible]	No 1		Cinders compacted	Metal pipe where necessary	6 ft. square	½ in	Float and trowel		Moist 4 days		5 in	
Mississippi	Jackson			3 in	1 in	3 sand 6 gravel slag broken stone or brick	1 or 2 sand as directed		Through 1½ in.					6 in clay pipe	Max. 30 sq ft		Trowel and smooth finish	Cuts between slabs				
Tennessee	Chattanooga	1913	2 in	4 in	1 in	3 sand [illegible] stone or gravel or 3 sand 5 limestone [illegible]	1½ sand or crushed gravel	¼ in to No 40	¼ to 1 in	M or H	No 1	Boxes	Cinders, slag or gravel			¼ in.	Float and trowel	⅛ to ¼ in metal at each section, ⅛ to ½ in every 50 ft.	Board canvas, etc. cover 5 days			
	Memphis	1916		4½ in 3½ in	1¼ in 1 in	2.7 sand 5.4 broken stone screened gravel 3.4 sand 6.8 broken stone screened gravel	3 crushed granite 1 sand 2 crushed limestone	Top ¾ to No 80 base ¼ to No. 100	¼ to 2 in	M or H No 1 2 turns dry, 3 wet	H	Boxes or barrels	Direct on subgrade		6 x 6 ft.	½ in.	Trowel smooth	⅛ in between slabs; 1 in. every 25 ft	From elements, moist 3 days			
	Nashville	1916	2 in	4 in	1 in	3 sand or limestone 5 crushed limestone or clean river gravel	1½ limestone screenings or 2 crushed gravel	¼ in to No 40	¼ to 1½ in	M	M or H No 1	1 cu ft boxes	Steam furnace cinders or gravel		As directed	¼ in.	Float, dressed and finished	⅛ in. every 25 ft.				
Arkansas	Little Rock			3 in	¾ in	7 clean sharp gravel	1½ torpedo sand						(Sent by letter. Nothing else specified.)									
Louisiana	New Orleans	1916		4 in	1 in	2 sand 6 gravel or stone	1½ sand		Through 2 in	M or H			Direct on subgrade				Troweled, not polished	¾ in every 30 ft		Street name 6 in high	4 in	1 in
	Shreveport		3½ in	3½ in	½ in	3 river sand 5 stone gravel or brick	1½ river sand			No 2			Gravel stone or brick, passing 2 in				Floated smooth, mark	1 to 1 in. at corners				
Oklahoma	Muskogee			3½ in	½ in	5 Ag. Ag—3 sand 5 stone or gravel	1½ sand	Clean and coarse	½ to 2 in	No 2 2 turns dry, 1 wet		Volume	Direct on subgrade		5 to 6 ft.		Trowel 3 times or more	1 in along curb and every 50 ft	Covered from weather 24 hours, closed 3 days		3½ in.	1 in.
Texas	Dallas	1916		3¼ in	¾ in	2½ sand or crushed stone 5 gravel or crushed stone	2 sand crushed stone or gravel screenings	¼ in to No 80	¼ to 1½ in	M or H No 1	H No 1	Boxes			5 x 5 ft	¼ in	Trowel to even surface	½ in every 25 ft and along curb	From weather, moist 4 days	Name month and year	6 in.	
	Houston	1916	4 in	4½ in	¾ in.	2½ sand 4 gravel or crushed rock	1½ sand or crushed rock screenings	Through ¼ in	¼ to 1½ in				Gravel, slag or cinders		6 ft		Trowel and brushed	⅛ in. every 6 ft., 1½ in. along curb ½ in every 36 ft.	From sun, moist 3 days	Name as directed	5 in	1 in
Arizona	Phoenix (1 course)	1911		4 in		2 sand 3 broken stone or screened gravel		Through ¼ in	¼ to 2 in	M or H No 1		Bag or wheelbarrow	Direct on subgrade				Trowel and smooth finish	⅛ in. every 15 ft., ¼ in. along curb	Covered with sand, earth, burlap, wet 7 days	At each lot		
	Phoenix (2 course)	1915		3½ in	½ in	3 sand 6 broken stone or screened gravel	2 sand	Through ½ in	¼ to 2 in	M or H No 1			Direct on subgrade				Trowel and float, smooth surface	⅛ in every 15 ft., ½ in. along curb	Covered with sand, earth, burlap, wet 7 days	At each lot		
Colorado	Colorado Springs		6 in or none	3¼ in	⅝ in.	3 sand 6 gravel or 7 pit run gravel	2 sand		¼ to 1½ in	M or H No 1			On subgrade 6 in gravel on adobe		6 ft square		Trowel smooth and true, steel trowel	½ in every 50 ft.	Canvas, etc., wet 7 days	At end of walk		
	Denver	1911	3 in	3¼ in	⅝ in.	3 sand 5 crushed river or screened gravel	2 sand	Through ½ in	¼ to 2½ in	M or H No 1 3 turns dry, 1 wet	No. 1 3 turns wet	Hopper, box or wheelbarrow	Direct on subgrade		7 ft.		Float and trowel, brushed	½ in. every 7 ft.	From injury, heat, cold or weather	Name of city and contractor 1 in long x ⅛ in. deep		6 in
	Pueblo	1911	3 in or none	3½ in	½ in on 6 ft. or 1 in on 6 ft or —	2 sand or crushed screenings 4 gravel screenings or crushed stone or 6 unscreened river gravel	2 sand or screenings	Through ¼ in	¼ to 1¼ in	M or H No 1		Boxes if directed	Cinders sand gravel, crushed stone through 2 to 3 in. on earth		6 ft		Trowel, roughened on over 5% grade	¼ in between blocks curb	From weather, moist 3 days	On each walk, ones includes date		
Idaho	Boise City	1913		3¼ in	¾ in	9 river run gravel	2 sand	Through ¼ in.	Through 2½ in	M or H No 1	No 1		Direct on subgrade		3 ft. square	¼ in.	Worked and rough surface	¼ in every 60 ft.	Cover, keep wet 48 hours, uncover and sprinkle, 7 days	Name, month and year stamped every 50 ft.	3½ in.	1½ in.
Montana	Butte	1915	4 in.	3 in	1 in	3 sand 5 crushed stone or slag or bank run gravel	2 sand	Through ½ in.	Through [illegible] in	No [illegible]			Cinders		5 ft long	⅜ in.	Trowel and float and rough or groove	Every 20 ft.		Name, date, street twice in block 1 x ⅛ in		
Utah	Ogden City	1916		3¼ in	¾ in.	2½ sand 5 gravel or broken stone	2 sand	Through ¼ in	¼ to 2 in	M			Direct on subgrade		12 ft long		Rough trowel, metal trowel to smooth surface	½ in. every 50 ft., 1 in. along curb	Burlap cover, wet 1 day, then sand, wet 7 days		5 in	1 in
	Salt Lake City	1916		3¼ in	⅜ in.	2 sand 4 crushed stone or gravel	2 sand	66% on No. 30, 93% on No. 100	¼ to 1 in	M or H No 1 3 turns dry 2 wet	M. or H. No 1	Wheelbarrow or machine	Direct on subgrade		12 ft. long		Smooth finish	Joint every 50 ft.	From sun, moist 7 days	Name and year twice in block 1½ to ⅛ in	4½ in	1½ in
Wyoming	Cheyenne		5 in	3¼ in	¾ in	3 sand 5 gravel or broken stone	2 screened sand			M or H	No 1	Boxes or barrels	Gravel or cinders		5 ft.		Float, 2 hours later wood trowel, groove, dot, smooth	Sand between slabs	From sun, kept wet several days			
California	Los Angeles	1910	2 in or none	3 in	½ in	3 sand or stone screenings 5 gravel or crushed stone	1½ sand	90% through No 4, 35 to 85% on No 30	¼ to ¾ in	M or H No 2 3 turns dry 2 wet	No 1 3 turns dry, 2 wet	Boxes	Direct on subgrade, 2 in stone or gravel on adobe				Float and trowel into body float on 5% grade	⅛ in roofing paper joint every 10 squares on adobe or new fill	From weather, wet 10 days	Every 100 ft.; letters ⅜ in. high and ⅛ in deep		
	Pasadena	1908	2 in or none	3 in	½ in	3 sand 6 crushed stone or gravel	2 sand		Pea to 2½ in	H No 2 2 turns wet	No 1		2 in sand on adobe		3 ft. square		Smoothly polished		From sun, watered 10 days			
	San Diego	1915	4 in or none	3 in	½ in	3 sand 5 screened gravel or broken stone	1½ sand		¼ to 2 in	M or H No 2 2 turns dry	No 1	Boxes or wheelbarrow	On adobe, 4 in damp sand		2 ft. square	3 in. curb to P L.	Float and trowel	Every 20 ft	2 in. earth, wet 7 days	Contractor's name 2 ft long; street name 3 ft. long		
	San Francisco			3 in	½ in	4½ gravel or stone sand to fill voids	1 gravel	Through ¼ in., 60% through No 20	¼ to 1¼ in	M or H No 1 3 turns wet	No 1	Boxes or mixing platform	Tamped or rolled, flooded if sand		3 ft square		Float, roughened on 12% grade	½ in filled hot asphaltic cement every 75 ft.	Cover of earth or sand 7 days wet 2 days	Street name		
Oregon	Portland			3 in	¾ in	3 sand 6 stone or gravel	1½ sand	Through ¼ in., 75% through No. 30	¼ to 2 in	M or H No 1	M or H No 1		Direct on subgrade		3 ft.		Trowel to grain	⅛ to 1 in strips every 12 ft	Sprinkle for 5 days	Name, date, street at corners		
	Salem			3½ in	½ in	To fill voids in sand and stone	1½ sand		Through 1½ in	M.	M	Boxes	Direct on subgrade		2½ x 2½ ft or as directed	¼ in	Troweled, roughened	¼ in heavy paper every 50 ft and against curb	Moist 3 days, in hot weather 7 days	Name of street at corners		
Washington	Spokane	1917	6 in	3½ in	¾ in	2½ sand 5 gravel or broken stone	2½ sand						On subgrade cinders sand, gravel if required		2½ x 2½ ft	¼ in.	Float, rough finish if required	⅛ in every 20 to 30 ft	Cover with sand, burlap, etc., moist 3 days	Name and date twice in block, 1 x ⅛ in.		
	Tacoma		6 in	3½ in	½ in.	3 sand 6 gravel	1½ sand 2 lb lampblack per barrel of cement	Through No 10, 8% through No 80	¼ to 2½ in.	M	M.	Boxes or frames	Proper material if subgrade is wet otherwise direct on subgrade		Approximate square	¼ in		¾ in every 25 ft	1 in sawdust, wet 7 days, and canvas in cold or rain		5½ in.	1 in
Connecticut	Hartford	1916	12 in	4 in	1 in.	2½ sand 5 broken stone	2 sand, or 1 screenings 1 sand	Clean, sharp	¼ to 1½ in	M or H		Boxes, pails, wheelbarrows	Cinders or gravel		4 to 6 ft		Troweled		From sun rain and freezing			
	New Haven	1910	12 in	3 in	1 in	2½ sand 5 gravel or crushed stone	2 sand		½ to 1½ in				Sand, gravel, cinders or crushed stone		5 x 6 ft		Troweled to smooth surface	Every 3[illegible] ft			4 in	1 in
Maine	Portland (1 course)	1917		4 in		2½ sand 4 crushed stone				M			Direct on subgrade				Float to smooth surface	¼ in as shown on plans	Traffic 7 days			
Massachusetts	Boston	1916	12 in	3 in	1 in	2 sand . 5 crushed stone	1 crushed trap or granite	Through ¾ in	¼ to 1¾ in	H No. 1			Stone, gravel, cinders	4 in. sewer pipe	Min 12 sq ft Max. 36 sq ft	⅛ in	Floated, toothed roller	½ in sand, each slab		Name and date at each end of walk	4½ in	1½ in
	Fall River	1917	12 in	3 in	1 in.	2½ sand or sand screenings 5 crushed stone	1 sand . 1 stone screenings	Through ½ in	¼ to 1¾ in	M.	H No 1	Mixing boxes	Coarse coal cinders			⅛ in.	Floated and troweled smooth polish, roughen on 4% grade	Sand between slabs, metal strips where and as directed	From sun, moist 3 days	At end of walk	4 in.	1½ in
	Lowell	1917	12 in	3 in	1 in	2 sand 5 crushed stone	Limestone dust		Through ¾ in				Clean cinders				Floated and troweled smooth, roughen, dot roller		Cover with sand, moist 4 days			
Rhode Island	Woonsocket	1916	0 in in rock, 12 in in earth	5 in	1 in	3 sand 6 crushed trap or screened gravel	2 sand or crushed trap	Through ½ in	¼ to ½ in				Cinders		6 ft	¼ in	Floated and troweled	¼ in. wide as directed	Covered 3 days kept moist	Name address date at end of each section		
New Jersey	Jersey City		6 in	4 in.	1 in	2 sand 5 gravel	2 sand	Fine to coarse	Small to medium	M or H. No 1	No 1		Cinders			½ in	Batters, 1 2 dry coat float, toothed roller	1 x 6 in white pine, 200 ft apart				
	Paterson		6 in	3 in	1 in	2½ sand 4 broken stone	3 Franklinite grits		Through 1½ in		No 1		Clean cinders		5 ft square	¼ in	Floated and troweled with brush	2 pieces of 3-ply roofing paper between slabs				
	Trenton	1914		3 in	1 in	3 sand 6 river gravel, crushed stone or broken stone	2 sand	Through ¼ in	¼ to 1¼ in	M or H No 1			Direct on subgrade		5 ft square		Trowel smooth, brush, finish with jointing tool		Moist 4 days from traffic 10 days			
New York	Albany		12 in	5 in	1 in	2 sand 4 broken stone or gravel	2 iron tailings and lampblack		¼ to 1 in	M	No 1		Cinders or gravel on clay	3 in clay pipe not used in sand			Screened, floated, grooved or marked	⅛ in between slabs	Cover with tar paper, etc., moist, for week after set			
	Borough of Brooklyn		6 in	3 in	1 in	2 sand 4 stone or gravel	1½ sand	⅛ in to No. 20	1¼ in to dust	M or H No 1		Boxes	Cinders and gravel through 1½ in		5 ft square			⅛ to ¼ in between slabs	From sun, moist until set			
	Borough of Bronx		5½ in	3½ in	1 in	3 sand 6 stone or gravel	2 gravel screenings		¼ to [illegible] in	M or H N[illegible]			Steam ash cinders		4 to 5 ft		Surface smooth, tooth roller	Sand every slab	From sun, frost cover moist 3 days			
	Borough of Manhattan	1914	7 in	4 in	1 in	2 sand 4 stone or gravel	1½ trap granite or grit	No 2[illegible] to No 4	¼ to 1¼ [illegible]	M or H N[illegible]		Receptacles	Gravel broken stone or steam ashes		6 ft			⅛ in each slab ½ in every 50 ft	Protect and moist for 3 days			
	Borough of Richmond (1 course)		12 in	4 in		2 sand 4 broken stone			1 in to dust	M or H			Clean gravel		4 to 6 ft		Trowel to smooth true finish	¼ in between slabs	From sun and kept moist until set			
	Buffalo		3 in	4 in	1¼ in.	6 gravel	2 gravel	Through ¼ in	2 parts through ¼ in	M	No 1	Boxes or barrels	Cinders or broken stone		4 ft	½ in	Floated or troweled	Cut through base at each slab	Tight covers 3 days			
	Rochester	1916	6 in	3 in	1 in	2 sand 4 stone or screened gravel	1½ sand gravel or screenings	Through ¼ in	Through ¾ [illegible]	No 1			Sand cinders gravel or stone		Generally 4 ft	⅛ in	Smooth surface and floated if required	Between slabs ½ in at all intersections	Covered and sprinkle 3 days, moist 3 days more		4½ in	1½ in
	Syracuse		4 in	3 in	¾ in	3 sand 6 gravel or broken stone	1½ sand or stone screenings	0.01 in to [illegible] past No 1[illegible]	[illegible]	N[illegible]	No 1	Receptacles	Broken stone cinders or gravel	4 in tile	5 ft max		Floated and troweled		Cover with canvas or wood frame until set	Contractor's name and year		
Pennsylvania	Erie	1915	12 in	[illegible] in	1 in	2 sand 5 gravel or stone	1 sand 1½ gravel or stone		Through 1 in				Sand gravel or hard cinders		5 x 6 ft	[illegible] in	Floated finished to grade	1 in every 11 ft along curb and [illegible]	Cover of wet paper and sand hot weather sand moist 4 days	Each end every walk		
	Philadelphia	1916	14 to 8 in, rock cut	3 in	1 in	3 sand 6 crushed stone or gravel	1 sand 1½ crushed gravel or stone	[illegible]	Through 1 in	M or H No 1 4 turns wet			Cinders		11 ft square	¼ in	Float and trowel, smooth surface [illegible]	[illegible] every 5 ft	Canvas or straw 3 days, sprinkled			
	Pittsburgh (1 course)	1916	5 in	5 in		2 sand 4 gravel or crushed stone		Through ¼ in	¼ to 1¼ in	M or H		Boxes	Cinders or clinkers ¾ to 1½ in through mesh	10 x 12 in every 25 ft as required	6 ft long	¼ in	Tamp, float roughen on steep grades	Sand each slab, ⅜ in every 50 ft	Moist 4 days protect against surface damage			
Illinois	Chicago		9 in	4½ in	¾ in	2½ fine aggregate 5 gravel, slag or broken stone	1½ fine aggregate	Sand or gravel screenings fine aggregate ¼ in to No [illegible]	[illegible]	M or H No 1	H in box		Cinders		Max 8 ft. sq	½ in	Troweled smooth			End of walk or front parcel land	7 in	2 in
	Joliet		12 in	3½ in	¾ in.	2½ torpedo sand 5 crushed limestone	1½ torpedo sand		¼ to 1 in	No 1	No 1		Cinders		6 ft square	[illegible] in		Every 5 ft through walk				
	Moline (1 course)			5½ in		1½ sand 3 gravel		¼ in to fine	¼ to ¾ in	M or H No 2			Direct on subgrade		[illegible] as walk		Tamped and troweled	[illegible] in between slabs	Covered moist 2 days			
	Peoria		4 in	3 in	1 in	3 sand 5 gravel	1½ sand			H	H		Cinders		4 x 6 ft		Float and trowel to smooth surface	[illegible] ¼ in wide every 4 ft				
	Springfield (1 course)	1916	4 in	4 in		1 sand 3 gravel			Through 1 in	No 1			Cinders tamped		5 ft	¼ in	Troweled brush finish	⅛ in every 40 ft	From traffic			
Indiana	Evansville		8 in	4 in	¾ in	2½ sand 5 broken limestone or gravel	1½ crushed gravel or quartz screenings	Through ¼ in	¼ to 1 [illegible]	M or H No 1	No 1		[illegible]		[illegible] x 6 ft [illegible] in grooves	¼ in	Trowel smooth	[illegible] in every 50 ft. and along curb	From traffic and weather, wet 3 days			
	Fort Wayne				1 in	[illegible] sand 5 gravel	1 sand		Through [illegible] in	M or H No 2	No 1		Gravel			¼ in	Trowel and floated smooth surface					
	Indianapolis	1914	12 in	3 in	1 in	[illegible] sand 5 gravel or stone	[illegible] sand	⅛ in to No 80	¼ to 1 in	M No [illegible]	M or H		Gravel, cinders and broken stone		6 ft		Troweled smooth and true	[illegible] in every 6 ft ½ in every 25 ft	From traffic and weather, moist 3 days			
	South Bend	1901		3¼ in	¾ in	1 bank run gravel	1½ sand		Through 1 in	N[illegible]						¼ in	Troweled or floated		Covered 3 days moist until set			
Michigan	Detroit	1912	6 in	3 in	1 in	3 sand 6 stone or gravel	2 sand or fine screenings		¼ to 1 in	M or H No 1			Cinders stone sand, ½ to 2 in			½ in.	Corrugated roller	1 in along curb and every 100 ft., ½ in between slabs	From weather and kept moist until set			
	Grand Rapids	1915		3½ in	½ in	2½ sand 5 gravel	2 sand	⅛ in to No 50	¼ to 1 in	M or H No 2					6 ft square		Troweled smooth, brush finish on 3% grades	¼ to ¾ in along curb	From weather, moist 3 days		5½ in	¾ in
	Saginaw	1915	4 in	3 in	1 in	3½ sand 5 stone	2 sand	Through ¼ in.	¼ to 1½ in	M or H			Gravel crushed stone sand screenings and cinders		5 ft	¼ in	Wooden float roughen	½ in every 50 ft.	From weather, moist 3 days	Name and year at each end		
Ohio	Cincinnati	1909	[illegible] in	3 in	1 in	2½ sand 5 stone or pebbles	1½ sand	Through ¼ in	¼ to 1½ in	M or H No 1			Clean soft coal cinders	3 in tile as directed	4 to 8 ft	⅛ in	Trowel smooth float on 4% grades	½ in. along curb, ¾ in every 30 ft			5 in	2 in
	Cleveland			3 in	1½ in	3 sand 6 gravel	2 sand	Through No 10 [illegible] through No 30	No 10 to 1 in	M or H No 1 [illegible] turns	No 1	Boxes	Sand gravel cinders		5 x 6 ft		Float or trowel as ordered	⅛ in sand each slab	From weather	Month and year on every block, contractor's name every 5th block	4 in	2 in
	Columbus		8 in	3 in	1 in	2 sand 4 stone or crushed gravel	1 granite screenings or 1 sand [illegible] torpedo sand	Through ¼ in	½ to 1 in	No 1	No [illegible]		Cinders	Subdrainage as directed	4 to 6 ft	½ in.	Float and trowel to smooth surface	¼ in between slabs	Covered 3 days moist	Month and year on every block	5 in	2 in
	Springfield		8 in	3 in.	1 in	5 clean gravel	1½ sand			No 1	No 1		Clean gravel		5 x 6 ft	¼ in	Float smooth coat 1 cement to 1 lake sand, float	Tar joint between slabs	From sun and wind until set			
	Toledo		3 in	3½ in.	1 in	2 sand 4 wash gravel or crushed stone	1 sand	Through ¾ in [illegible] through No [illegible]	¼ to ¾ in	M			Cinders or coarse sand		Max. 6 ft. Min. 4 ft		Float and trowel groove checker broom on steep slopes	⅛ in tar paper or sand between slabs	From sun, and sprinkle 4 days	Name and date every 50 ft	4 in	2 in.
	Youngstown		4 in.	3 in	1 in	2 sand 4 gravel or limestone	2 sand		Through ¾ in				[illegible]	3 in pipe	4 to 6 ft	⅛ in	Stroke, float trowel rough, groove on slope	Strip of paper between slabs				
Wisconsin	Madison		7 in	4 in	1 in	3 sand 5 broken stone	1½ crushed gravel screenings		Through 1½ in				Gravel or cinders		As directed		Brush or corrugate					
	Milwaukee	1917		4 in	1 in	2½ sand 5 gravel [illegible] or slag	2 sand		¼ to 1 in	M or H No 1	M or H No 1	Wheelbarrows etc	Direct on subgrade		6 ft		Trowel smooth	⅛ to ¼ in between slabs, 1 in. along curb	From weather, moist 3 days	Name, date, city, at ends of work	6 in	2 in.
	Oshkosh	1909	4 in	4 in.	1 in	3 sand 6 stone or gravel	1½ sand						Cinders			¼ in					5 in.	1½ in.
	Racine	1906	2 in in sand, 6 in in clay	3 in	1 in	3 sand [illegible]	[illegible]				[illegible] 1		[illegible]		[illegible] sq ft	¼ in	Float and trowel [illegible] toothed roller [illegible]		From weather, and kept moist until set		4½ in	1½ in.
	Superior		8 in	3 in	1 in	[illegible]	[illegible] gravel or clean sand		[illegible]	M or H	M or H		[illegible]	3 in pipe	[illegible] ft long		Trowel after until set	¼ in every 3 or 4 ft	From sun and kept moist until surface set		6 in	1 in
Iowa	Cedar Rapids (1 course)		5 in	4 in residence, 6 in business		1 [illegible]			[illegible]	N[illegible]		[illegible]	[illegible] stone	[illegible] tile	[illegible] ft long	¼ in	Trowel smooth finish	Every 50 ft ¼ in every slab	Cover until set ⅛ in sand moist 4 days			6 in.
	Des Moines	1917	4 in	3¼ in	¾ in	3 sand [illegible] gravel		Through [illegible] in	[illegible]	M			[illegible]		Not over [illegible] sq ft	½ in	Wooden float	1 in pitch along curb and as directed	Cover and moist until set from traffic		5¼ in	¾ in
	Sioux City (1 course)			4 in.		[illegible] screenings 4 gravel		Through [illegible] in	[illegible]	[illegible]			[illegible]		[illegible] ft		Float 1 1 dry mix float toothed roller	Every 50 ft				
Kansas	Topeka	1916		3[illegible] in	¾ in	[illegible] sand 4 [illegible] crushed limestone	1 [illegible]	[illegible]		M			[illegible]		Max 4 ft	¼ in	Trowel, glazed finish	½ in between slabs ½ in along curb	[illegible] moist as directed			
	Wichita			3½ in	½ in. and 1 in	[illegible] sand [illegible] 7 gravel	[illegible]			[illegible]			[illegible]		[illegible] ft square	¼ in	Trowel	¼ in every 25 ft and along curb	Kept moist [illegible] weather if necessary	Date, name both ends, letters 1½ in high		
Missouri	Kansas City	191[illegible]		[illegible] in	¾ in	1 sand 1 [illegible] Joplin flint [illegible] sand 4 crushed limestone	1 [illegible]	Through [illegible]		N[illegible]			[illegible]		[illegible] ft long	[illegible]	Trowel [illegible]	False joint every section [illegible]	[illegible] moist	Street name at corners	4½ in	¾ in
	St. Joseph			3 in	1 in	[illegible] limestone	[illegible] river sand			N[illegible]	1		[illegible]		[illegible]		[illegible]	[illegible] around lot and curb	Covered [illegible], moist			

[illegible] No. 1 [illegible]
No. 2 [illegible]
No. 3 cement and sand [illegible] stone [illegible]

Top No. 1 Cement and sand [illegible] cement and sand or screenings wet.
No. 2 [illegible]
cement and sand of screenings through No. 4 [illegible] cement and sand or screenings [illegible]

latter type. Walks heaved by frost in many instances, but return to proper grade later. Do not retain as uniform surface, however, as cinder subbase walks.

Ogden, Utah.—Walks laid directly on subgrade. Very little trouble due to freezing. Experience throughout state is that artificial subbase unnecessary.

Norfolk, Va.—Sandy soil with drain tile to sewer system gives good drainage. Frost not over 6 in. Have experienced entire satisfaction in omitting subbase.

Spokane, Wash.—Very few places where subbase has been used and although frost heaves some walks out of grade in winter, these settle back to normal level subsequently with little or no injury. Natural drainage good and in only a few cases have tile drains and gravel subbase been used.

Salt Lake City, Utah.—Soil gravelly clay to clayey loam. City has, almost without exception, laid walk directly upon natural ground. Subbase a needless expense where natural ground is or can be put in good, stable condition.

PROPOSED REVISED SPECIFICATIONS FOR CONCRETE SIDEWALKS.

These specifications apply to concrete sidewalks in residence and business districts, and cover the preparation of the subbase, the laying and finishing of the walk and its protection during early hardening

GENERAL REQUIREMENTS.

MATERIALS.

1. *Cement.*—The cement shall meet the requirements of the current Standard Specifications for Portland Cement of the American Society for Testing Materials.

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable, crushed rock or gravel consisting of quartz grains or other hard material graded from fine to coarse, with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other organic matter nor more than three (3) per cent, by weight, of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered at any time during progress of the work. If there is more than seven (7) per cent of clay or loam, by volume, in one (1) hour's settlement after shaking in one hundred (100) per cent excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquets or cylinders, shall show a tensile or compressive strength, at seven (7) and twenty-eight (28) days, at least equal to the strength of briquets or cylinders composed of one (1) part of the same cement and three (3) parts standard Ottawa sand by weight. The percentage of water used in making the test specimens of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand test specimens of standard consistency. In other respects all briquets or cylinders shall be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials. (See Cement Specifications, A.S.T.M.)

(*b*) Coarse Aggregate: Coarse aggregate shall consist of clean, durable, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one (1) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

(*c*) No. 1 Aggregate for Wearing Course: No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter and shall contain no soft, flat or elongated particles. It shall pass, when dry, a screen having one-half ($\frac{1}{2}$) in. square openings, and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

3. *Mixed Aggregate.*—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

4. *Subbase.*—Only clean, durable material, such as coarse gravel or steam-boiler cinders free from ash or particles of unburned coal shall be used in the subbase. (NOTE.—Eliminate this clause unless subbase is required.)

5. *Water.*—Water shall be clean, free from oil, acid, alkali, vegetable or other organic matter.

6. *Color.*—If artificial coloring material is required only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement.

7. *Reinforcement*—The reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft.

8. *Joint Filler.*—The joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather, or prepared strips of fiber matrix

and bitumen as approved by the architct or engineer. The strips shall be one-half (½) in. in thickness, their width shall at least equal the full thickness of the slab and their length shall at least equal the width of the slab at the joint.

SUBGRADE.

9. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The subgrade shall be thoroughly tamped until it is brought to a firm, unyielding surface. It shall have a slope toward the street curb of not less than one-half (½) in. per ft.

When the concrete sidewalk is to be constructed over an old path composed of gravel or cinders, the old path shall be entirely loosened, the material spread for the full width of the subgrade and compacted as specified.

10. *Deep Fills.*—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited. The top of all fills shall extend beyond the walk on each side at least one (1) ft., and the sides shall have a slope greater than one (1) on one and one-half (1½).

11. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the architect or engineer.

12. *Depth.*—The subgrade shall be not less than ——— (—) in. below the finished surface of the walk.

NOTE.—Subgrade to be at least five (5) in. below the finished surface of the walk when subbase is not required and at least eleven (11) in. below when subbase is required.

SUBBASE.

(Omit these sections unless subbase is required.)

13. *Thickness.*—On the subgrade shall be spread a material as hereinbefore specified which shall be thoroughly rolled or tamped to a surface at least ——— (—) in. below the finished grade of the walk. On fills, the subbase shall have the same slope as the sides of the fill.

14. *Wetting.*—While compacting the subbase, the material shall be kept thoroughly wet, and shall be wet when the concrete is deposited, but shall show no pools of water

FORMS.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the walk

17. *Division Plates.*—Suitable metal divison plates shall be provided to completely separate adjacent slabs during construction unless otherwise permitted by the architect or engineer.

18. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION,

19. *Size of Slabs.*—The slabs or independently divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft., nor shall the length of any such slab be greater than one and one-half (1½) times the width. Larger slabs shall be reinforced as hereinafter specified.

20. *Thickness of Walk.*—The thickness of the walk shall not be less than ——— (—) in.

NOTE.—Walks for residence districts to be at least five (5) in. thick and for business districts six (6) in. Where walks cross driveways in residence districts, the total thickness shall be increased to six (6) in. with a one (1) in. wearing course.

21. *Joints.*—A one-half (½) in. joint shall be provided at least once every fifty (50) ft. in the length of the walk which shall be filled with suitable material as specified under "Joint Filler." A similar joint shall be provided at each intersection of sidewalk and street curb and at such other points as may be designated by the architect or engineer. Sidewalks in business districts shall be separated from abutting buildings by a one-half (½) in. joint.

22. *Protection of Edges.*—Where required by the architect or engineer, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius one-half (½) in. The edges of all slabs abutting a business street which act as curbing must be protected by suitable metal angles or corner-bars as approved by the architect or engineer.

MEASURING AND MIXING.

23. *Measuring.*—The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered one (1) cu. ft.

24. *Machine Mixing.*—All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of any approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all materials are in the drum. The drum shall be completely emptied before receiving material for the succeeding batch.

25. *Hand Mixing.*—When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

26. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, remixing with or without additional materials or water, shall not be permitted.

PROTECTION.

27. *Treatment.*—As soon as the finished walk has hardened sufficiently to prevent damage, the surface of the walk shall be sprinkled with clean water or preferably covered with at least one (1) in. of wet sand or earth and kept wet for at least seven (7) days.

28. *Protection.*—The freshly finished walk shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by rain drops, and the contractor shall provide and use, when necessary, sufficient tarpaulins to completely cover all sections that have been placed within the preceding twelve (12) hours. The contractor shall erect and maintain suitable barriers to protect the walk from traffic and any section damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at this own expense in a manner satisfactory to the architect or engineer. Before the sidewalk is opened to traffic the covering shall be removed and disposed of by the contractor. The walk shall not be opened to traffic until the architect or engineer so directs.

29. *Temperature Below 35 Degree Fahrenheit.*—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will, within twenty-four (24) hours, drop to 35 degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

30. *Name Stamp.*—The contractor shall mark in the walk at each end of every one hundred fifty (150) ft. of walk or fraction thereof, either by stamping or by inlaying an approved metal plate, his name and the year in which the walk is constructed. The stamped letters shall be one (1) in. high and one-fourth ($\frac{1}{4}$) in. deep. If a metal plate is used, the top of the plate shall be flush with the top of the walk.

TWO-COURSE SIDEWALK.

For two-course walks the following will apply in addition to the general requirements:

CONCRETE BASE.

31. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half ($2\frac{1}{2}$) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

32. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar

33. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the walk. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly-laid concrete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen subgrade or subbase.

34. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal. The reinforcement shall not be less than one (1) in. from the finished surface of the walk.

WEARING COURSE.

35. *Proportions for Mixture No. 1.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement and two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters ($\frac{3}{4}$) in. (NOTE.—Proportions and thickness for residence districts or where traffic is light.)

36. *Proportions for Mixture No. 2.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in. (NOTE.—Proportions and thickness for business districts or where traffic is heavy.)

37. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard

38. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

39. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening. When division plates

are not used, the slab markings shall be made in the wearing course directly over the joints in the base with a tool which will completely separate the wearing course of adjacent slabs. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

40. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

ONE-COURSE SIDEWALK.

For one-course walks the following will apply in addition to the general requirements:

41. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

42. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strikeboard. The consistency shall not be such as to cause a separation of the coarse aggregate from the mortar in handling.

43. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly-laid concrete. In no case shall concrete be deposited upon a frozen subgrade or subbase.

44. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall be placed two (2) in. below the finished surface of the walk. The reinforcement shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

45. *Finishing.*—After the concrete has been brought to the established grade by means of a strikeboard, it shall be floated with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel troweled, but excessive working shall be avoided. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-alf ($\frac{1}{2}$) in.

PROPOSED REVISED SPECIFICATIONS FOR CONCRETE FLOORS.

These specifications apply to floors in buildings, whether subjected to moderate or heavy traffic, and cover the laying and finishing of the floor; also its protection during early hardening.

For architects, engineers and others desiring to embody these specifications in their general specifications covering a particular piece of work, the following outline of the paragraphs necessary to meet different conditions will prove convenient:

Floors Laid on Ground.—Moderate or Light Traffic.

Two-Course.—Paragraphs 1–15 (except 2*c*); 30–47; 49–52.

One-Course.—Paragraphs 1–15 (except 2*c* and *b*); 30–42; 53–57.

Floors Laid on Ground.—Heavy Traffic.

Two-Course.—Paragraphs 1–15; 30–46; 48–52.

Reinforced Concrete Floors.—Moderate or Light Traffic; Paragraphs 1–22 (except 2*c*); 24–29.

Heavy Traffic; Paragraphs 1–23; 25–29.

GENERAL REQUIREMENTS.

MATERIALS.

1. *Cement.*—The cement shall meet the requirements of the current Standard Specifications for Portland Cement adopted by the American Society for Testing Materials.

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested, and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, crushed rock or gravel, consisting of quartz grains or other hard material, graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other organic matter nor more than three (3) per cent by weight of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered at any time during progress of the work. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in one hundred (100) per cent excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquets, shall show a tensile strength at seven (7) and twenty-eight (28) days at least equal to the strength of briquets composed of one (1) part of the same cement and three (3) parts Standard Ottawa sand, by weight. The percentage of water used in making the briquets of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquets of standard consistency. In other respects all briquets shall be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials. (See Cement Specifications, A.S.T.M.)

(*b*) Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one and one-half ($1\frac{1}{2}$) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed

(*c*) No. 1 Aggregate for Wearing Course: No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. It shall pass when dry a screen having one-half ($\frac{1}{2}$) in. openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

3. *Mixed Aggregate.*—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

4. *Subbase.*—Only clean, hard material, such as coarse gravel or steam-boiler cinders, free from ash or particles of unburned coal, shall be used in the subbase. (NOTE.—Eliminate this clause when subbase is not required.)

5. *Water.*—Water shall be clean, free from oil, acid alkali or vegetable matter.

6. *Color.*—If artificial coloring matter is required, only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement

7. *Reinforcement.*—The reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond.

8. *Joint Filler.*—The joint filler shall be a suitable compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather; or, prepared strips of fiber matrix and bitumen as approved by the architect or engineer. The strips shall be one-half ($\frac{1}{2}$) in. in thickness and their width shall at least equal the full thickness of the slab.

MEASURING AND MIXING.

9. *Measuring.*—The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform

proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered as one (1) cu. ft

10. *Machine Mixing.*—All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all the materials are in the drum. Raw materials shall not be permitted to enter the drum until all the material of the preceding batch has been discharged.

11. *Hand Mixing.*—When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

12. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, remixing with or without additional materials or water, shall not be permitted.

PROTECTION.

13. *Treatment.*—As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor shall be covered with at least one (1) in. of wet sand, or two (2) in. of sawdust, which shall be kept wet by sprinkling with water for at least ten (10) days.

14. *Protection.*—The freshly-finished floor shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by raindrops, and the contractor shall provide and use when necessary sufficient tarpaulins to completely cover all sections that have been placed within the preceding twelve (12) hours.

15. *Temperature Below 35 Degrees Fahrenheit.*—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will, within twenty-four (24) hours, drop to 35 degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

REINFORCED-CONCRETE FLOORS.

For reinforced-concrete floors the following will apply in addition to the general requirements:

16. *Forms.*—The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours, and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the architect or engineer in charge. When freezing weather occurs, the supports shall remain in place an additional time, equal to the time the floor has been exposed to freezing.

17. *Reinforcement.*—Reinforcing metal shall be provided as called for on the plans. It shall be placed as indicated and mechanically held in position

so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at points of maximum stress shall be avoided. Splicing by lapping bars without contact and with space between bars along the overlap equal to twice the thickness of the bars is preferable to mechanical splices or clamps.

CONCRETE SLAB.

18. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate and four (4) cu. ft. of coarse aggregate.

19. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that may be readily caused to flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

20. *Placing.*—The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. Concrete shall be deposited in its full position as soon as possible after mixing and within thirty (30) minutes after the water has been added to the dry materials. It shall be struck off to a surface at least one (1) in. below the established grade of the finished surface of the floor. Workmen shall not be permitted to walk in freshly-laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

21. *Joints.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the architect or engineer; joints to be vertical.

WEARING COURSE.

22. *Proportions and Thickness* (Mixture No. 1).—The mortar shall be mixed in the proportions of one (1) sack of portland cement, and two (2) cu. ft. of fine aggregate. The maximum thickness shall be three-quarters (¾) in.

23. *Proportions and Thickness* (Mixture No. 2).—The mortar shall be mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

24. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

25. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard.

NOTE.—When placing the wearing course after the concrete slab has hardened, eliminate paragraph 25 and substitute the following:

26. *Preparation of Slab.*—The surface of the slab shall be thoroughly roughened by picking, and swept clean of all dirt and débris.

27. *Placing.*—The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed. A neat cement

grout shall be brushed on the surface of the slab, the wearing course immediately applied and brought to the established grade with a strikeboard. Grout and mortar shall be used within forty-five (45) minutes after mixing with water.

28. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired.

29. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

PLAIN CONCRETE FLOORS.

For plain concrete floors the following will apply in addition to the general requirements:

SUBBASE.

30. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The subgrade shall be thoroughly tamped until it is brought to a firm, unyielding surface.

31. *Deep Fills.*—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited.

32. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

33. *Depth.*—The subgrade shall be not less than ———— (00) in. below the finished surface of the floor.

NOTE.—Subgrade is to be five (5) in. below the finished surface of the floor when subbase is not required, and at least eleven (11) in. below when subbase is required.

SUBBASE.

(Omit these sections when subbase is not required.)

34. *Thickness.*—On the subgrade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled or tamped to a surface at least ———— (00) in. below the finished grade of the floor. On fills, the subbase shall extend the full width of the fill.

35. *Wetting.*—While compacting the subbase, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

FORMS.

36. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

37. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the floor.

38. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

39. *Size of Slabs.*—The slabs or independently-divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft. Larger slabs shall be reinforced as hereinafter specified.

40. *Thickness of Floor.*—The thickness of the floor shall be not less than five (5) in.

41. *Width and Location of Joints.*—When required by the architect or engineer in charge, a one-half ($\frac{1}{2}$) in. space or joint shall be left between the floor and the walls and columns of the building, to be filled with the material before specified under "Joint Filler."

42. *Protection of Edges.*—Where required by the architect or engineer in charge, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

TWO-COURSE FLOOR.

CONCRETE BASE.

43. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half ($2\frac{1}{2}$) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

44. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

45. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established elevation of the floor. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly-laid con-

crete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen subgrade or subbase.

46. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric, or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

WEARING COURSE.

47. *Proportions for Mixture No. 1.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters ($\frac{3}{4}$) in.

48. *Proportions for Mixture No. 2.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement and one (1) cu. ft. of fine aggregate, and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

49. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

50. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

51. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

52. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

ONE-COURSE FLOOR.

53. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

54. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strike-board. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

55. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly-laid concrete. In no case shall concrete be deposited upon a frozen subgrade or subbase.

56. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having any dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

57. *Finishing.*—After the concrete has been brought to the established grade by means of a strikeboard, and has hardened somewhat, but is still workable, it shall be floated with a wood float in a manner which will thoroughly compact it and provide an even surface. When required, the surface shall be steel-troweled, but excessive working shall be avoided. Unless protected by metal the surface edges of all slabs shall be rounded one-half (½) in.

REPORT OF THE COMMITTEE ON NOMENCLATURE.

The committee has held no meetings during the year. War activities have so completely taken the time of the chairman that committee work has had to suffer. Most of the other members of the committee could undoubtedly say the same.

About a year ago a working policy was agreed upon and a program of work laid out. An unofficial ballot asking for approval or disapproval and criticism of the individual definitions of the 1917 report of the committee was sent out to all members of the Institute and to a selected list of non-members.

In submitting the 1917 report it had been recognized by the committee that there was much room for improvement and since discussion of nomenclature reports is not allowed on the floor of most conventions this method was taken of bringing together carefully considered discussion.

Fifty-two replies were received all containing valuable suggestions and criticisms. A digest of these criticisms was prepared by the Secretary and if this digest could be published, the members of the Institute would have a better idea of the difficulties which confront a nomenclature committee.

From the 1917 report and the criticisms received a revision of the definitions proposed has been made by the chairman and the secretary. This new set definitions, however, has not been submitted to the other members of the committee and we consequently do not ask for publication of the revised definitions at this time.

The chairman asks that the committee be continued for another year during which time we can give the members of the committee opportunity for full consideration of a revised report and to go as much further along the lines already laid down as the exigencies of war time will permit.

W. A. Slater,
Chairman.

REPORT OF THE JOINT COMMITTEE OF THE AMERICAN CONCRETE INSTITUTE AND PORTLAND CEMENT ASSOCIATION ON CONCRETE BARGES AND SHIPS.

The idea of building ships or other floating vessels of reinforced concrete is not new. For many years and in several different countries, attempts have been made from time to time to build small boats and barges of reinforced concrete. From the information at hand, apparently some of these attempts have been successful and the vessels thus built have been in use for considerable periods. However, no definite information regarding boats built prior to the war is at hand which would assist your committee in solving the general problem of the concrete ship.

Since the beginning of the war, however, owing to the loss of the world's tonnage due to submarine sinkings, the attention of many naval architects, concrete engineers and others has been drawn to the question of concrete ships to replace those sunk. The scarcity of steel, wood and labor, and the length of time necessary to construct ships of steel or wood directs attention generally to the substitution of reinforced concrete. Norway appears to have taken the lead in this work and two different companies are already building ships of concrete. The Porsgrund Cement Works, whose vice-president and general manager, Jens Hauland, has recently been in this country, has already launched one or more ships of 100-tons cargo capacity and is reported to have under construction a ship of 1000-tons cargo capacity. The general design of these ships follows generally that of a framed steel ship, and your committee has been informed by Mr. Hauland that the weight compares very favorably with that of a steel ship. No definite information is available, however, by which this statement can be verified.

The Fougner Steel Concrete Shipbuilding Co. of Christiania, Norway, has been building vessels since June, 1916. About eighteen in all have been launched up to the present time. Several others are under construction. The "Namsenfjord", about 400-tons displacement, launched some time ago, has made a round trip between Norway and England. No detailed information is available at the present time regarding these vessels that would throw light on the general problem.

On this side of the Atlantic, at least two ships are under construction. At San Francisco, a large ship, about 5000-tons capacity, is being built and, from information at hand, will be shortly launched. Your committee learns that this ship is 320 ft. long, 44 ft. 6 in. beam and 30 ft. deep. At 24-ft. draft the displacement is said to be 8000 tons. The weight of the hull is said to be 2200 tons. At Montreal, Canada, a small concrete ship which will have about 300 tons carrying capacity has already been launched and is now being equipped. This vessel is being constructed by the Atlas Construction Co., Ltd., of which Chas. M. Morssen is president. Your committee is indebted to Mr. Morssen for considerable data relating to this ship and it takes this opportunity of acknowledging the very courteous treatment shown its representatives by Mr. Morssen and the freedom with which he discussed the details of the design and construction. This ship is 126 ft. long, between perpendiculars, 22 ft. 6 in. beam, with a depth of 12 ft. 6 in. The displacement is about 650 tons. The ship will be self-propelled and is now being equipped with boilers and engines. She will shortly make her first trip. No estimate of cost is at present available.

With the exception of the two ships noted above, little information has been gained from the ships now under construction which will assist in the solution of the concrete ship problem.

The present report will confine itself to a general statement of the several elements which make up the concrete ship problem and a discussion of the information obtained from the tentative design of a concrete ship.

INFORMATION NEEDED.

In order to make any advance toward the solution of the concrete-ship problem, information must be obtained concerning several points regarding which only meager information is now available. These points taken together constitute the concrete-ship problem.

(1) *The Relation Between Carrying Capacity and Displacement.*—The displacement of a ship is the weight of the water she displaces, and is therefore, the sum of the weight of the ship itself and its cargo capacity expressed in tons. The cargo capacity will hereafter be spoken of as the "dead weight"—following the usual practice with naval architects. Wherever displacement, dead weight or weight of ship is spoken of, it will be in terms of long tons (2240 lb.) which is the usual practice.

It is apparent that the efficiency of a ship as a cargo carrier depends upon the relationship between dead weight and displacement. Expressed in terms of per cent, in the average cargo ship built of steel, the dead weight is from 70 to 75 per cent of the displacement—taking into account as weight of ship all spars, fittings, deck houses, anchors and chains, auxiliary engines and tanks, but not boilers, engines or coal. In a wooden ship, the dead weight is from 60 to 65 per cent of the displacement. It is quite evident that from the difference in weight of materials, it will be difficult to design a ship of concrete that will give a relationship between dead weight and displacement approaching that of steel. However, if ships are to be built of concrete for commercial use, the weight of the ship must be such as to provide a reasonable dead weight or cargo capacity for the displacement.

(2) *Transverse Strength.*—The stresses in the transverse members of a ship are, in still water, functions of the draft and the stiffness, and may be computed by mathematical processes, although the computations are long and laborious. When the material is reinforced concrete, the problem becomes much more complicated. Experience has shown, however, that numerous elements other than draft effect the transverse strength of a ship, such as the effect of rolling in a seaway, impact with docks or other ships and stresses incident to going into dry-dock. The transverse members of cargo ships of

today are, therefore, not designed to withstand computed stresses, but are designed in accordance with various rules which embody the result of long experience in the construction and use of ships. It should be noted in this connection that granting of insurance depends on compliance with these rules.

Steel ships are of two different types (*a*) framed ships in which transverse ribs or frames are spaced from 18 to 24 in. on centers, the plating being riveted to these ribs without intermediate longitudinal members, excepting in the bottom, and (*b*) longitudinally framed ships (Isherwood) in which heavy frames are spaced from 10 to 15 ft. on centers with intermediate longitudinals to which the plating is riveted.

From a comparison with the ordinary steel ship design, it would appear to be not difficult to design transverse members of reinforced concrete of equivalent strength to steel members—the question of strength only being considered.

(3) *Longitudinal strength.*—A ship must be able to meet conditions which are unlike any to which land structures are subject.

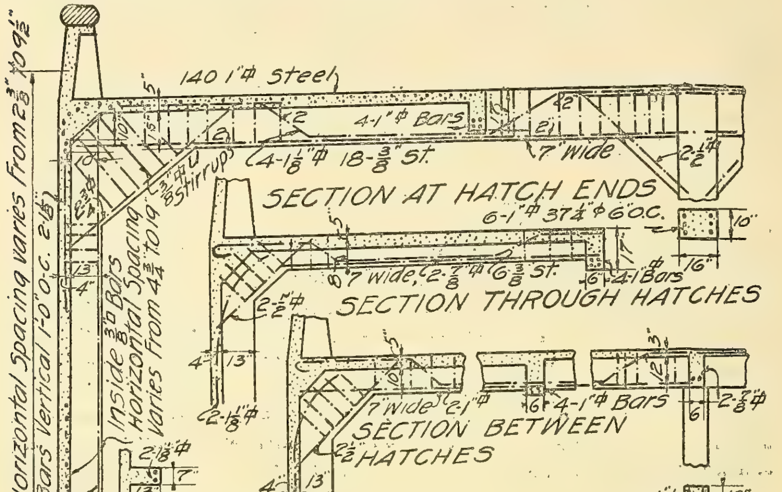
In determining the longitudinal strength of a ship, it is customary to assume two conditions. Under the first condition, the ship is assumed to be suspended between two wave crests, the length between crests being equal to the length of the ship between perpendiculars, the height of the wave being equal to one-twentieth of that length. In this case, the ship, as a whole, is acting as a simple beam supported at the ends. This condition is termed "sagging." Under the second condition the ship is assumed to be supported amidships on one crest of the same wave. Under this condition, the ship as a whole acts as a cantilever. This condition is termed "hogging." It is apparent therefore, that when a ship is riding the waves both the deck and the bottom of the ship will be required to withstand tensile and compressive stresses alternately—the maximum tensile stress following the maximum compressive stress at very short intervals. In a steel ship the entire cross-sectional area of the midship section acts to resist these stresses, taking into account, in determining the moment of inertia, all of the continuous members, such as continuous scantlings and deck, side and bottom plates. In the concrete ship, equivalent strength must be provided. In the case of the concrete ship, however, only the steel reinforcement can be relied upon to take tensile stresses. The concrete assisted by the steel, will take the compressive stresses.

The effect of the rapid change of the character of the stress in either the deck or the bottom is much more serious in the case of concrete ship then in the steel ship for the reason that, owing to the low tensile stress of concrete, cracks will occur at the extreme fiber under relatively low tensile stresses in the steel. These cracks, if any, alternately opening and closing, may tend to cause disintegration, with resulting leaks or impairment of the reinforcement.

At the present time, little information is available as to the effect of such reversal of stress, and but little can be hoped for until an actual trial has been made of a concrete ship in a sea.

(4) *Elasticity.*—There is an almost unanimous opinion among naval architects and seafaring men generally that a concrete ship will be so inelastic

that she will tear herself to pieces in a sea. While it is doubtless true that on a concrete ship there will not be the same readjustment of stresses as in a steel ship when subject to the action of a heavy sea, experience with reinforced-concrete structures generally has shown that such structures have considerable elasticity and there is ample reason for the hope that reinforced concrete will prove a suitable material for ship building purposes.



SOME DETAILS OF THE PROPOSED DESIGN FOR SEAGOING CONCRETE BARGE SUBMITTED BY THE JOINT COMMITTEE ON CONCRETE SHIPS AND BARGES OF THE AMERICAN CONCRETE INSTITUTE AND PORTLAND CEMENT ASSOCIATION.

(5) *Effects of Sea Water on Concrete and Reinforcing Steel.*—Until very recently little information has been available as to the effect of sea water on concrete. The recent work of the Bureau of Standards, acting in cooperation with the Portland Cement Association, has thrown considerable light on what may be expected from the action of sea water. The result of their investigation tends to show that inferior concrete or concrete of which the surface skin has been impaired suffers serious disintegration when in contact with sea water. Inasmuch as in most instances the structures examined, which form the basis of the report of the Bureau of Standards, were built without thought as to the action of sea water (it being assumed that there would be no deleterious action), there is every reason to hope that where the effect of sea water is appreciated, and where steps are taken in the way of selected materials and adequate workmanship, assuring a good mix and a satisfactory surface skin, the concrete will not so deteriorate.

With regard to the effect of sea water on the reinforcing steel, there is some question as to whether a thin layer of concrete can be relied upon to protect the steel from corrosion. To provide a thick protective layer of concrete outside of the reinforcing steel is practically out of the question, owing to the large increase in weight. If the reinforcement, therefore, is to be maintained in perfect condition, the steel must be protected by galvanization and by increasing the efficiency of the protective concrete by means of additional care in materials and workmanship and by a surface coating of a waterproofing character.

(6) *Relative Cost.*—Just at the present time, the demand for tonnage is so great that any ship of reasonable capacity that can be used for carrying cargo will prove financially successful. The relative costs of ships of concrete and steel, or concrete and wood, is not therefore as important a consideration as it will be after the war when conditions again approach the normal. However, it is necessary to have an adequate idea of the probable cost of a concrete ship as well as a comparison with the cost of steel and wooden ships.

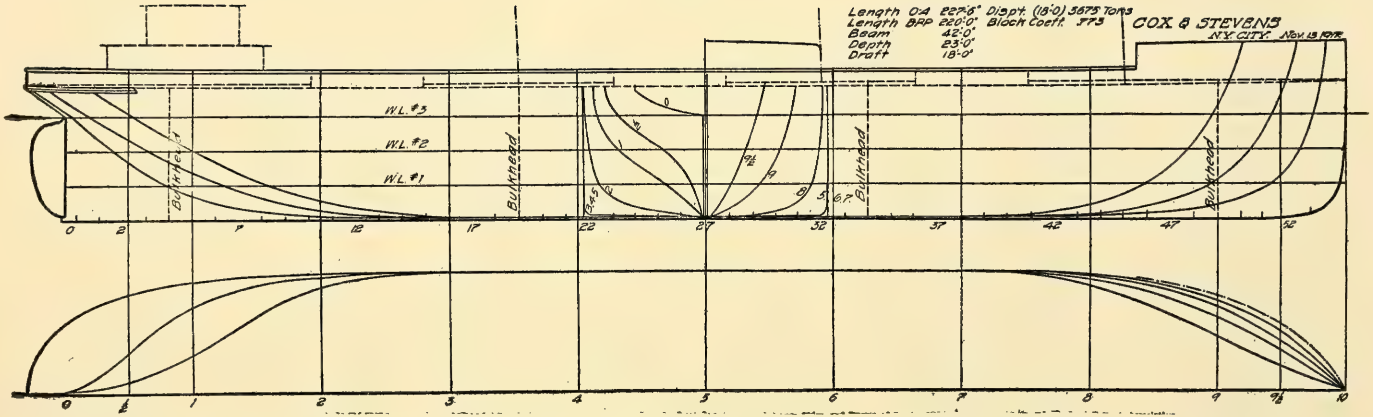
(7) *Speed of Construction.*—Speed of construction is of vital importance in the ship building program today owing to the immediate requirements for tonnage. If it shall be found that the concrete ship can be constructed in much less time than a steel or wood ship, this fact will contribute largely to the success of the concrete ship.

TENTATIVE DESIGN FOR A CONCRETE BARGE.

Boats or ships devoted to carrying of cargo may be divided, for the purpose of this report, into two general classes: those adapted to use in still waters, such as harbors, rivers and canals; and those adapted for use at sea, in which provision must be made for wave action.

Your committee devoted much thought and study to the question of what type of ship would be most suitable for study at this time.

Ships for use in still water do not present the same problems of construction as do ships for ocean service. Owing to the need of barges of from 100 to 150 ft. in length for use on the canals of New York State, it was your



SUGGESTED LINES FOR 2,000-TON SEAGOING REINFORCED-CONCRETE BARGE, ACCOMPANYING REPORT OF THE JOINT COMMITTEE ON CONCRETE SHIPS AND BARGES OF THE AMERICAN CONCRETE INSTITUTE AND PORTLAND CEMENT ASSOCIATION.

committee's first thought to devote study at this time to the design of a barge which would be suitable for this service. After considerable study and discussion with various naval architects, including the firm of Cox & Stevens of New York, who were retained by your committee in an advisory capacity, it was deemed advisable to abandon the study of a small still-water barge for the reason that while such a study might result in considerable information as to the best method of construction and dead weight compared to displacement, the problem which is of vital importance to solve at this time—namely whether or not concrete can be adopted for ships necessary for transatlantic and coastwise service as well as possible service on the Lakes—would remain unsolved, and little or no information would be obtained bearing on this phase of the question. It was therefore decided to devote the study to a ship of the sea-going type. In order to simplify the problem as far as possible, without omitting any essential elements, it was decided to select a barge to be towed rather than a self-propelled ship. This would eliminate all consideration of weight of all boilers and engines excepting such as would be required for auxiliary hoisting machinery. Because of the fact that so little is known regarding the concrete ship, it was deemed advisable to keep the ship as small as possible. It was thought at first to limit the size to an assumed carrying capacity of 1000 tons or less.

After many conferences by your committee with its consulting naval architects, it was decided to increase the capacity to about 2000 tons, it being the decided opinion of the naval architects that a ship smaller than 2000 tons would have little commercial value after the war, although during the war it would probably prove successful.

In the absence of any reliable data as to the relation between dead weight and displacement for concrete ships, an assumption was made that the dead weight would be approximately 55 per cent of the displacement for the proposed tentative barge. Upon this supposition the consulting naval architects laid out the lines for the proposed ship. The dimensions are as follows:

Length (over all)	227 ft. 6 in.
Length between perpendiculars	220 " 0 "
Beam	42 " 0 "
Depth	23 " 0 "
Loaded draft	18 " 0 "

The hull was given lines such as would provide maximum carrying capacity and still permit reasonable towing and adequate control. At an 18-ft. draft the displacement will be about 3675 tons, at 19-ft. draft, the displacement will be about 3860 tons.

The general dimensions of the barge, and the lines are indicated on the blue print herewith attached. It should be noted that the midship section of the hull is rectangular with the exception of a slight crowning of the deck and a slight inclination of the bottom upward toward the bilges (dead rise). The deck is straight throughout and continuous from stem to stern. The ship is divided into five compartments by four transverse bulkheads, the foremost and aftermost compartments are to be used for tanks and ballast, while the three center compartments are for cargo. . The cargo compartments served by three hatches—each about 14 x 19 ft.

Design.—As has already been noted, the actual stresses in the structural members of a cargo ship are not usually computed, but sections are adopted which long experience has shown to be safe.

After consultation with the naval architects, it was thought desirable to take as a criterion a steel ship designed in accordance with the rules as laid down by Lloyds and provide equivalent strength in reinforced concrete with such modifications as the difference in material would make necessary. Assuming that the concrete for the ship would be poured as in a concrete building, a limit of three inches was placed upon the thickness of the concrete, as that was considered the minimum thickness that can be poured satisfactorily.

The question of weight being vital in the design of a concrete ship, it was deemed desirable to assume a concrete of high crushing and shearing strength. A concrete of one part portland cement and one part carefully selected sand and two parts selected gravel (about ½-in.) was decided upon, which if properly mixed should give an ultimate crushing strength in excess of 3000 lb. per sq. in. The stress in the concrete in extreme fiber was limited to 1000 lb. per sq. in. In order to reduce the danger of cracking the concrete in extreme fiber, a large percentage of steel was assumed with a resulting low tensile stress.

For a ship of the same size Lloyds' rules require that the ribs or frame be placed 24 in. on centers. This was deemed too close for economical use of concrete and a spacing of 4 ft. was adopted, making the resisting moment of each frame at all points equivalent to twice the resisting moment of one frame in the steel ship. In determining the strength of the frames of the steel ship as fixed by Lloyd's rules, no allowance was made for the effect of the tank top (floor of the hold) or of the bottom plating. Assuming a spacing of frames of 4 ft. and a resisting moment equivalent to two frames, under Lloyd's rules the floor plate or transverse girder supporting the ship's bottom was found to require a concrete girder 36 in. deep and 10 in. wide including the bottom slab.

To provide for the negative and positive bending moment in the shell, two lines of reinforcement was assumed—one near the outer surface and one near the inner surface. The load being uniform, the positive bending moment at the center between frames will be approximately one-half of the negative bending moment at the frames. Two-thirds of the total amount of steel was, therefore, provided near the outer suface and one-third near the inner surface, all the bars being straight, it being considered impractical to bend the bars. From the lower turn of the bilge up to the deck, one size of bar was assumed with variable spacing to meet the required strength at different depth. The thickness of the shell from the upper turn of the bilge to the rail was fixed at 4 in. To provide adequate strength against "hogging" or "sagging," tension steel to the extent of 140 sq. in. was provided in both deck and bottom. This steel was considered also as providing reinforcement for local stresses in the bottom due to hydrostatic pressure and in the deck due to load. It was placed in two layers to take up both negative and positive moments, two-thirds being near the outside surface and one-third near the inner surface. Since this steel will be subject to compresion as well as tension the two layers were securely tied together to prevent buckling. To provide adequate protection to the steel and also to provide sufficient area in concrete to take up the compression stresses, the thickness of the bottom up to the upper turn of the bilge was made 5 in. The deck between the bulkwark and the line of the hatch coamings was made 5 in. also. The deck between hatches and within the lines of the hatch coamings was made 3 in. thick. The deck itself not being sufficient to provide sufficient area in concrete, additional area was added at the junction of the deck and the sides where it would also tend to increase the resistance to horizontal shear which would be high at that point.

In the bottom, three longitudinal members, running from bow to stern, were provided—one on the center line and one at each lower turn of the bilge. These longitudinals or "keelsons" give longitudinal stiffness, assist in distributing the load due to dry-docking, and provide additional area for hogging and sagging stresses.

The vertical frames in the sides were made 7 in. wide and 17 in. deep, including the shell and an equivalent in strength to steel frames under Lloyds' rules. At the junction between the floor plate and the side frames, a heavy gusset is provided to give additional strength at that point. A smaller gusset is provided between the side ribs and the deck beam. This gusset is increased in size at the beams at hatch ends. The deck along the center line is supported by means of posts at the ends of each hatch and at alternate deck beams between hatches.

The bulkheads consist of 4-in. slab reinforced near both surfaces and stiffened by reinforced ribs 20 in. in depth over all and 7 in. wide, spaced 4 ft. on centers.

The general features of the concrete design are indicated on the drawing herewith.

Estimate of Quantities and Weights.—A conservative estimate of quantities required for the proposed barge gives the following:

Concrete	731 cu. yd.
Steel	482,000 lb.
Yellow pine timber for floor of hold	30,000 ft. b.m.
Oak timber for fender and rail	15,000 ft. b.m.

Taking the weight of concrete at 144 lb. per cu. ft., weight of steel 490 lb. per cu. ft. and weight of wood 48 lb. per cu. ft., and making an allowance of 77 tons for deck machinery, anchors and chains, spars, water tanks and deck houses, the total weight of ship would be 1647 tons. Taking the displacement of 18 ft. draft at 3675 tons, the carrying capacity or dead weight would be 2028 tons. Assuming 19-ft. draft, the displacement would be 3860 tons and the dead weight capacity 2213 tons.

Cost.—Assuming sand, stone, cement and reinforcing steel at current prices, the cost of the hull (exclusive of any equipment such as deck houses, spars, anchors and chains, engines, boilers and tanks) would approximate $118,000. Assuming the construction of ways which could be used for the construction of five or more ships, the cost of ways per ship would approximate $8000, making a total cost of the hull (exclusive of equipment) $126,000.

Based on the figures noted above, the cost of the hull per ton of dead weight would approximate $63. Owing to a wide range in costs of steel and wooden ships of the same character, it is somewhat difficult to arrive at anything approaching a definite estimate for steel or wood ships. The best figures available seem to indicate that the cost of steel hull per dead-weight ton ranges from $90 to $120, while the cost of a wooden hull ranges from $70 to $100.

OTHER TYPES OF CONCRETE SHIPS.

The designs of various other types of ship in which reinforced concrete has been proposed either alone or in combination with structural steel were examined by our committee.

In some of these designs cellular sides and bottoms are proposed. Although double side and bottoms in ships have distinct advantages, when the material is concrete, such a design appears out of the question not only because of the considerable increase in weight, but also because of the added complication to the form work, already a large item in the cost of a concrete ship.

Designs have been proposed in which structural steel members are used as reinforcement. While such members assist somewhat in maintaining the lines of the ship during the construction of the form work, the added advantages can hardly offset the increased cost due to the fabricated steel work and the inefficient use of the reinforcement.

One design has been proposed in which structural steel members are used to support a shell of reinforced concrete. While the claim is made that a ship can be built of the proposed design with only about one-half the steel used in a steel ship of the same capacity, and at much greater speed of construction, the difficulty of making the two materials act together, thus avoiding cracks, makes the design as proposed of doubtful value.

Your committee feels that it is not its function at this time to prepare any detailed plans for a concrete vessel. It is manifestly impossible to design a type of vessel that will be applicable to all classes of service. A barge for use on the canals of New York would not be an efficient type for use in harbors or on the Ohio or on the Mississippi.

Vessels heretofore built have demonstrated that the small barge for still-water service can be built and successfully operated. The solution of the larger problem of a concrete ship will include the solution of smaller vessel problem in which questions of strength are not of the same prominence.

Although there are some questions regarding the concrete ship which can only be answered by actual experiment, the studies which your committee has made point to the commercial success of the concrete ship.

Your committee suggests that specifications for a concrete vessel should embody the following principles:

1. Both cement and aggregates should be selected with great care to insure a concrete of maximum efficiency.

2. The concrete should be placed in one continuous operation to insure monolithic construction. The concrete mixture should be such as will develop a crushing strength in excess of 3000 lb. per sq. in. when tested in standard cylinders at 28 days. A concrete consisting of 1 part portland cement, 1 part sand and 2 parts ½-in. aggregate may be expected to give such a concrete. The mixture and workmanship in placing must be such as will assure impermeability.

3. The reinforcing steel should be in the form of deformed bars and should be galvanized.

4. In parts of the vessel where cracks in the concrete would tend to cause leaks, the stress in the steel should be kept low (preferably less than 12,000 lb.).

5. Some form of elastic waterproofing coating should be applied to the hull below the deck.

COMMITTEES ON CONCRETE BARGES AND SHIPS.

AMERICAN CONCRETE INSTITUTE.

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November 27, 1917.

ADDENDA TO THE REPORT OF THE JOINT COMMITTEE OF THE CONCRETE INSTITUTE AND THE PORTLAND CEMENT ASSOCIATION ON REINFORCED-CONCRETE SHIPS AND BARGES.

Since your committee prepared the above report there has been a rapid development of the concrete ship problem. This development has been mainly due to the fact that early in January the United States Shipping Board Emergency Fleet Corporation organized a Department of Concrete Ship Construction for the investigation of the concrete ship problem, and the design and construction of concrete ships.

The program of the Department of Concrete Ship Construction includes the design and construction of harbor and sea-going tugs, 7500-ton tankers, 7500 and 3500-ton cargo ships. One cargo ship of 3500 tons deadweight is under construction. In cooperation with the Inland Waterways Committee of the Railroad Administration, the department is designing barges for the New York State Barge Canal and other inland waters.

Since the preparation of the report, the "Faith," the 4500-ton ship of the San Francisco Shipbuilding Co., has been successfully launched, and has also passed successfully her trial trip. The 126-ft. cargo ship built by the Atlas Construction Co. of Montreal in the fall of 1917 has been completed and is about ready for service after having been frozen in the ice of the St. Lawrence River all winter.

June 25, 1918.

REPORT OF THE COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

The committee this year presents for the approval of the convention to be sent to letter ballot of the Institute certain corrections in Standards No. 5, 17, 18, 19 and 20, the various Standard Specifications on concrete roads and pavements.*

It also presents for the consideration of the Institute a Recommended Practice for Concrete Road and Street Construction, which is printed, beginning on the next page.

H. E. Breed, *Chairman.*

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*These specifications are printed on p. 573 of this volume.

RECOMMENDED PRACTICE FOR CONCRETE ROAD AND STREET CONSTRUCTION.

Submitted by the

COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

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CHAPTER I.

MATERIALS.

1. LABORATORY CONTROL.

Laboratory control, in order to obtain the greatest benefit, should be interpreted in a broad sense. Samples of all materials for use in concrete pavements should be tested in the laboratory. These samples should be representative of the supply, which necessitates some control over methods of sampling; also some provision should be made for field testing. This latter is essential especially on the fine aggregate, to insure that the material actually being used is practically as good a quality as the sample tested.

It is most advisable that all construction of concrete roads and pavements should be done under laboratory control. This control should extend to the testing, sampling, and a certain amount of field inspection, of all concrete materials. Testing and sampling are to be done prior to the acceptance of materials for use in any concrete road or pavement.

The object to be accomplished by laboratory control is to determine, prior to use, whether or not the materials available are of proper quality; also this would enable the laboratory representative to determine and suggest the proportions which would give the best results. All laboratory control must be based on certain tests of the constituent materials, of the finished concrete and an interpretation of such results.

Cement.—The American Concrete Institute has previously adopted the Standard Specifications for Portland Cement of the American Society for Testing Materials.

Fine Aggregate.—Fine aggregate when used in the wearing course should be of a high grade and, although thickness of concrete and other factors would have to be regulated to meet traffic on different highways, the fine aggregate should be of the same class when used for surfacing on any concrete highway.

Fine aggregate should consist of hard, durable particles smaller than one-quarter inch in size. No tests have yet been perfected to determine the durability of fine aggregates and this will have to be determined by visual inspection.

We recognize that the present tests and test methods leave much to be desired. It is recommended that the following tests be made to determine suitability of fine aggregate for use in concrete roads and pavements:

1. Organic impurities (colorimetric tests).
2. Sieve analysis.
3. Mortar strength test.
4. Volume of silt.

Organic Impurities.—It is a known fact that organic impurities which may be encountered in a few sands are likely to give poor results if such sand is used in concrete. A method for determining the existence of harmful organic impurities has been developed by Committee C-9 on Concrete and Concrete Aggregates, of the American Society for Testing Materials.

This appears in Part I of the 1917 Proceedings of the American Society for Testing Materials, pages 327-333, and is also printed in pamphlet form as Circular No. 1 of the Structural Materials Research Laboratory, Madison and Robey Streets, Chicago. Two methods for making the tests are described, namely, laboratory and field methods. The method for field tests, which in general are sufficient for this purpose, is given below.

Method for Field Tests.—"Fill a 12-oz. graduated prescription bottle to the 4½-oz. mark with the sand to be tested. Add a 3 per cent solution of sodium hydroxide (NaOH) until the volume of the sand and solution, after shaking, amounts to 7 oz. Shake thoroughly and let stand over night. Observe the color of the clear supernatant liquid.

"In approximate field tests it is not necessary to make comparison with color standards. If the clear supernatant liquid is colorless, or has a light yellow color, the sand may be considered satisfactory in so far as organic impurities are concerned. On the other hand, if a dark-colored solution, ranging from dark reds to black, is obtained, the sand should be rejected and should be accepted only after it has passed the usual mortar strength tests.

"Field tests made in this way are not expected to give quantitative results, but will be found useful in:

"1. Prospecting for sand supplies.

"2. Checking the quality of sand received on the job.

"3. Preliminary examination of sands in the laboratory.

"An approximate volumetric determination of the silt in sand can be made by measuring or estimating the thickness of the layer of fine material which settles on top of the sand. The percentage of silt by volume has been found to vary from 1 to 2 times the percentage by weight."

Sieve Analysis.—The proper gradation of fine aggregate has a direct influence on the strength of concrete. A sieve analysis is useful in arriving at the proper proportions of the fine and coarse aggregate to be used in the mixture. There is at present no uniform practice in the number and size of sieves used for this purpose. Five or six sieves ranging from 100 mesh to 4 mesh are sufficient for this test. It is recognized that the present practice of placing definite limitations on the percentage of a sample passing certain sieve sizes is open to criticism. However, the limitations given in the current specifications for concrete roads and pavements of the American Concrete Institute would give satisfactory results, although certain sands which may not fall within these limitations might, if properly proportioned with coarse aggregate, also give satisfactory results.

Mortar Strength Tests.—These are made to determine the hardening qualities of fine aggregate when mixed with portland cement. It is recommended that this test be made on all samples of fine aggregate. The following methods of tests are recommended:

Compression test on 2-in. cubes or 2 × 4-in. cylinders, on a mixture of 1 part cement to 3 parts of sand by either weight or volume. The volumetric method is considered more rational, since it approximates field conditions of proportioning. Parallel tests should be made by the same operator on standard Ottawa sand mortar, using the same cement. The test specimens should be stored under the same conditions.

As a basis for acceptance, the following relative strengths are recommended:

AGE OF TESTS.	MINIMUM STRENGTH OF FINE AGGREGATE MORTAR.
3 days.	1.15 times standard Ottawa sand mortar strength.
7 days.	1.05 " " " " " "
28 days.	1.00 " " " " " "

Silt Tests.—An excess of silt contained in the sand, although it may not affect the strength of the concrete, has a tendency to rise to the surface, resulting in a scaling of the top. Volumetric tests for silt determination are recommended. Although this method cannot be performed with the same degree of accuracy as the weight method, it can be more easily checked in the field.

As for the value of the above tests, organic impurities are detrimental to any sand for use in concrete. Gradation has a direct influence on the strength of concrete. An excess of silt is likely to result in a scaly surface. Organic impurities and gradation affect the mortar compression, and the latter might be considered a check on both organic impurities and gradation. The effect of the silt is one that can be determined in most cases only after concrete has been under service.

Coarse Aggregate.—One method of test, known as the Deval abrasion test, has been standardized and is at present in more or less general use for the testing of stone for all types of highway work. This has proved somewhat satisfactory for the testing of stone for macadam roads. It is questionable just how accurate it is for determining the suitability of coarse aggregate, both stone and pebbles, in concrete work. However, it is a fact that this test will indicate the difference in poor and good stone; and since as yet there are no suitable substitutes it is recommended that this test be used for determining the suitability of stone for use as coarse aggregate.

Several methods have been devised for the testing of pebbles to determine their suitability as coarse aggregate in concrete, but none of these are at present in general use. A recently designed improvement on the standard Deval stone abrasion machine, has the standard cylinder replaced by one that is slotted, thereby eliminating the dust cushion. This latter machine is at present being used in the New York State Highway Department with good results in the testing of slags and pebbles, which have never given satisfactory results in the standard cylinder.

Hardened Concrete.—Sampling and testing of concrete specimens cast in the field from material being placed is a valuable test not only on the aggregate being used but also on the manipulation. We recommend that provisions be made in the specification for the sampling and testing of field concrete. Specimens should be 6 inches in diameter and 12 inches long. Methods for making field tests may be found in the report of Committee

C-9 on Concrete and Concrete Aggregates of the American Society for Testing Materials, Part I of the 1917 Proceedings, pages 322-326. A pamphlet containing this report may be obtained from the chairman of the Sub-Committee on Field Tests, William M. Kinney, 210 South La Salle Street, Chicago.

Sampling.—Representative samples of the supply of coarse or fine aggregate proposed for use in concrete roads and pavements shall be submitted to the laboratory for test. These samples should be taken by an engineer, preferably one who is familiar with sampling materials, so that he will know how to obtain a representative sample. Various methods have been developed for sampling fine aggregate, but these are so numerous—varying as the condition of the supply varies—that it would not be practicable to enumerate or describe them. Great importance should be attached to the proper sampling of all materials, especially fine aggregate. Full information should be furnished with these samples, giving the extent of the supply, and proportions of different kinds of materials which may occur in the supply. A test of any material is only as valuable as the sample is representative of the supply. Samples of fine aggregate should consist of one-half cubic foot of material passing ¼ in. If run of product is sent in as a sample from a gravel bank containing particles above ¼ in., the sample should consist of ¾ to 1 cu. ft. of material. Samples of stone for use as coarse aggregate should be taken from an unweathered portion of the ledge and should contain 30 lb. of material.

2. CEMENT.

Only portland cement should be used in concrete pavements. It should meet the requirements of the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials. One sack of portland cement (94 lb. net) should be considered one cubic foot.

3. AGGREGATES.

Durable, clean, well-graded aggregates are absolutely essential to the success of a concrete pavement. Uniform quality is of great importance. Samples of the materials proposed for use should be submitted to the engineer before orders are placed and tests should be made on these samples to see that the specifications are fulfilled, although this is not so essential if aggregates from the source of supply in question are well known to be satisfactory and uniform in quality. Field tests for grading and cleanness should be made on shipments as delivered and frequent laboratory tests are recommended.

The different aggregates should be kept clean and separate until placed in the mixer. When aggregates are placed directly on the subgrade care must be exercised by the shovelers so that earth from the subgrade is not taken up with the aggregate.

Mixed aggregates such as bank-run gravel or crusher-run stone should not be used. Artificially prepared mixtures should not be allowed, since they will not be uniformly graded after handling, even if well graded when mixed.

Under rigid laboratory control it is possible to vary the proportions of aggregates and cement so that a strong concrete is obtained with materials not well graded. For the usual mix of fixed proportions, however, it is very important that fine aggregate be well graded with the coarser particles predominating, and that coarse aggregate be made up principally of the larger particles. Absence of the finer material in coarse aggregate is not detrimental; rather it is to be desired.

4. FINE AGGREGATE.

Cleanness of sand is essential for good concrete. By cleanness is meant freedom from clay, loam, silt, or organic material. Organic material is especially harmful; its presence often cannot be detected by inspection. Sand should consist of quartzite grains or other equally hard material; particles of mica and feldspar are injurious and are to be avoided.

Where stone screenings are used, freedom from dust must be insisted upon, as dust makes difficult proper mixing and will reduce the strength of concrete, unless the time of mixing is much increased.

Field tests for grading should be made with a nest of standard sieves. Volumetric tests for silt determination are recommended, in which sand and water are shaken together in a graduated glass cylinder. The depth of the deposit on top of the sand is a measure of the foreign material present.

Detection of organic impurities of a humus nature in sand (this includes practically all organic material found in natural sands) has been made easy by the recent development of the colorimetric test whereby the presence of these impurities can be detected in the field within 24 hours after the sand is delivered on the job.

(See recommendations under LABORATORY CONTROL.)

5. COARSE AGGREGATE.

Crushed stone should be tough, fairly hard and free from dust. There is no advantage in using coarse aggregate decidedly harder than the mortar. Limestone with a French coefficient of wear as low as seven, has been used successfully in one-course pavements. Brittleness is a quality to be avoided.

Where pebbles are to be used, attention should be directed toward detecting the presence of soft, flat or elongated particles. Aggregate containing such material should be rejected. Coarse aggregate containing particles up to three inches in diameter is being used satisfactorily in many localities.

6. QUANTITIES.

The quantities of materials for pavements of various thicknesses and widths are given in the table on page 525.

7. WATER.

Ample supply of clean water, free from oil, acid, alkali or vegetable matter, is necessary. Water containing a large amount of organic material should not be used.

8. JOINT FILLER.

Prepared strips of bituminous material and fiber matrix ¼ to ½ in. in thickness are recommended for expansion joints. The width of the joint filler should be at least ½ in. greater than the thickness of the pavement at any point. A joint filler which will not bend easily when concrete is deposited against it is desirable.

For contraction joints of the concealed type, strips of sheet steel 1/32 in. in thickness, 1 in. less in width than the least thickness of the pavement, are recommended. Two-ply tar paper has been used successfully for contraction joints. The paper should be brought to within ½ in. of the pavement surface.

9. PROTECTION PLATES.

Joint protection plates are not recommended.

10. REINFORCEMENT.

A coating of light rust on the reinforcement will not be detrimental to satisfactory results, but no excessive rust, paint, or other coatings should be present to interfere with proper bond. Care should be exercised to see that the reinforcement is so stored prior to use, that it is not coated with mud or clay when placed in the pavement. Reinforcement left on a job when contract is not completed at the end of the season should be collected and stored so that it will be protected from the elements. Wire mesh, expanded metal or rods may be used.

CHAPTER II.

DESIGN.

11. CROSS-SECTIONS.

General designs for cross-sections of concrete roads are illustrated in Fig. 1, which shows sections for single- and double-track roads under varying typical conditions.

12. WIDTH.

In the design of a road there are two essential dimensions—width and thickness—which must be determined and which are the chief factors controlling the cost per unit of length. As it is always the endeavor to make this cost as little as possible, too many roads are made both too narrow and too thin.

As the thickness of the pavement is to a certain extent a function of the width, the width is the dimension to be first determined. Owing to the greatly increased weight of traffic and to the fact that a large proportion of the traffic on the main roads is self-propelled, it becomes necessary that the width of the paved portion of the road should be sufficient to make it unnecessary for any of the wheels of a vehicle to leave the pavement and run upon the soft side or shoulder. If the traffic is all in one direction, and not so heavy but that a single track will be sufficient to care for it, then the paved portion of the road need be no more than is required for a single vehicle. This is a condition, however, that is never experienced practically,

TABLE 1.—AREA OF CROSS SECTION, CUBIC YARDS,* QUANTITIES OF MATERIALS REQUIRED PER LINEAR FOOT AND SQUARE YARDS OF SURFACE FOR CONCRETE ROADS FOR VARIOUS WIDTHS AND THICKNESSES SHOWN.

Width, Feet.	Thickness.			Area of Cross-section, Square Feet.	Cubic Yards Concrete per Linear Foot of Pavement.	Square Yards, per Mile.	Cement, Barrels.		Sand, Cubic Yards.		Rock or Pebbles, Cubic Yards.	
	Sides, Inches.	Center, Inches.	Average, Inches.				1:2:3	1:1½:3	1:2:3	1:1½:3	1:2:3	1:1½:3
9	6	8	7.333	5.500	.204	5,280	.355	.389	.106	.086	.157	.173
10	6	8	7.333	6.111	.227	5,867	.394	.433	.118	.095	.175	.123
18	6	8	7.333	11.000	.407	10,560	.708	.777	.212	.171	.313	.346
18	7	9	8.333	12.500	.463	10,560	.806	.884	.241	.194	.357	.324
18	8	10	9.333	14.000	.519	10,560	.903	.991	.270	.218	.400	.441
20	6	8	7.333	12.222	.453	11,733	.788	.866	.235	.190	.349	.385
20	7	9	8.333	13.777	.510	11,733	.887	.974	.265	.214	.393	.434
20	8	10	9.333	15.555	.576	11,733	1.002	1.100	.300	.242	.444	.490
24	6	8	7.333	14.667	.543	14,080	.945	1.037	.282	.228	.418	.462
24	7	9	8.333	16.667	.617	14,080	1.074	1.178	.321	.259	.475	.524
24	8	10	9.333	18.667	.691	14,080	1.202	1.320	.359	.290	.532	.587
26	6	8½	7.667	16.611	.615	15,253	1.070	1.175	.320	.258	.474	.523
26	7	9½	8.667	18.777	.695	15,253	1.208	1.327	.361	.292	.535	.591
26	8	10½	9.667	20.944	.776	15,253	1.350	1.482	.404	.326	.598	.660
27	6	9	8.000	18.000	.667	15,840	1.161	1.274	.347	.280	.514	.567
27	8	11	10.000	22.500	.833	15,840	1.449	1.591	.433	.350	.641	.708
30	6	9	8.000	20.000	.741	17,600	1.289	1.415	.385	.311	.571	.630
30	8	11	10.000	25.000	.926	17,600	1.611	1.769	.482	.389	.713	.787
36	6	9½	8.333	25.000	.926	21,120	1.611	1.769	.482	.389	.713	.787
36	8	11½	10.333	31.000	1.148	21,120	1.998	2.193	.597	.482	.884	.976
40	6	10	8.667	28.888	1.070	23,467	1.862	2.044	.556	.449	.824	.910
40	8	12	10.667	35.555	1.317	23,467	2.292	2.515	.685	.553	1.014	1.119

Cement for 1 cu. yd. concrete { 1-1½-3 Mix. : 1·91 bbls; 1-2 -3 Mix. : 1.74 bbls.
Sand for 1 cu. yd. concrete { 1-1½-3 Mix. : 0.42 cu. yds.; 1-2 -3 Mix. : 0.52 cu. yds.
Stone for 1 cu. yd. concrete { 1-1½-3 Mix. : 0.85 cu. yds.; 1-2 -3 Mix. : 0.77 cu. yds.

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*Based on 1 bbl. cement equal 4 cu. ft.: voids in stone, 45 per cent. Factors used from Taylor and Thompson "Concrete, Plain and Reinforced."

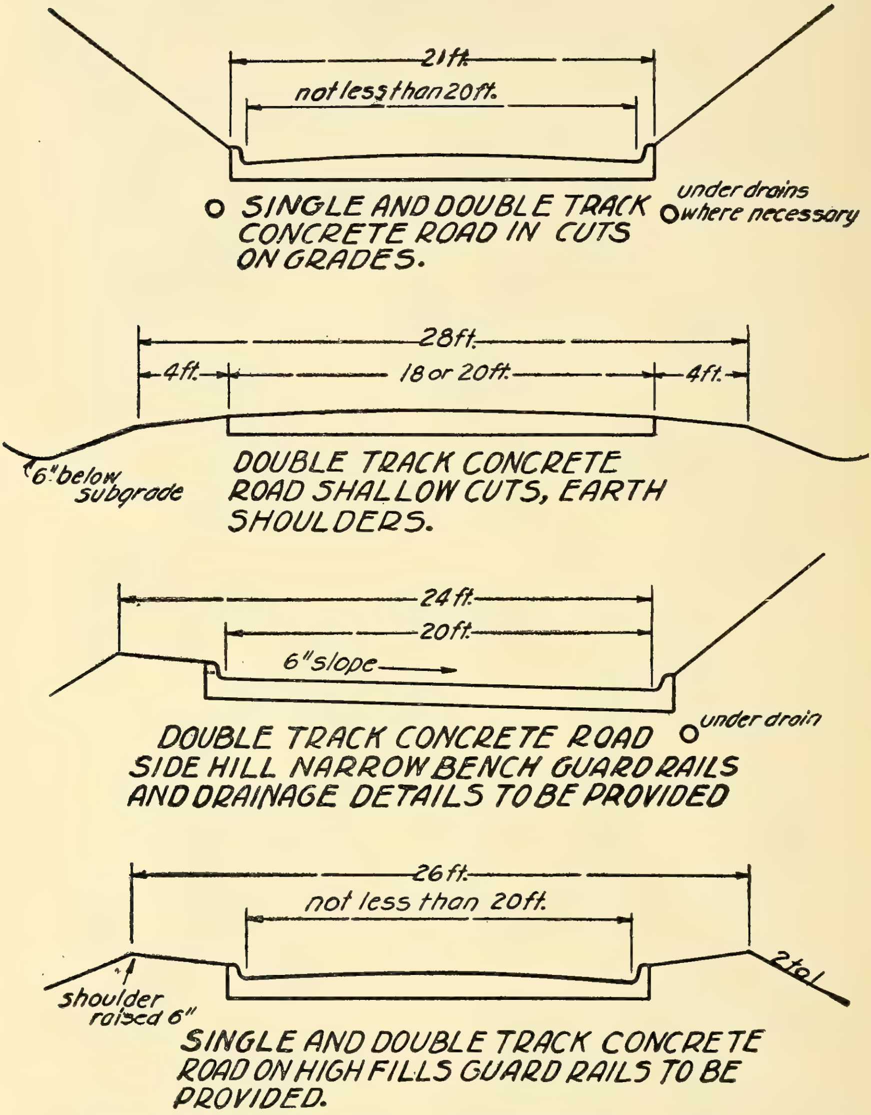
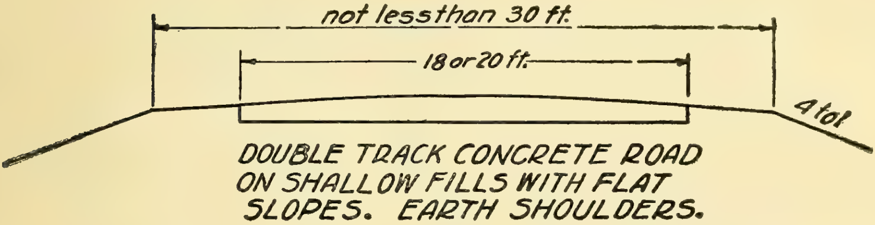
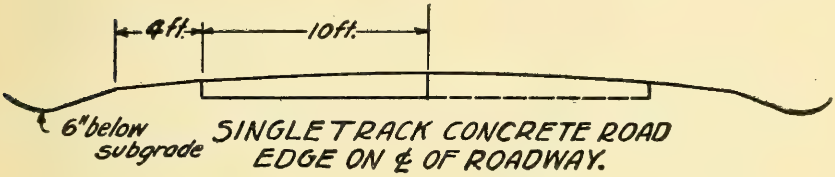

21 ft.
not less than 20 ft.
under drains where necessary
SINGLE AND DOUBLE TRACK CONCRETE ROAD IN CUTS ON GRADES.
28 ft.
4 ft.
18 or 20 ft.
4 ft.
6" below subgrade
DOUBLE TRACK CONCRETE ROAD SHALLOW CUTS, EARTH SHOULDERS.
24 ft.
20 ft.
6" slope
under drain
DOUBLE TRACK CONCRETE ROAD SIDE HILL NARROW BENCH GUARD RAILS AND DRAINAGE DETAILS TO BE PROVIDED
26 ft.
not less than 20 ft.
2 to 1
shoulder raised 6"
SINGLE AND DOUBLE TRACK CONCRETE ROAD ON HIGH FILLS GUARD RAILS TO BE PROVIDED.

FIG. 1.—RECOMMENDED CROSS-SECTIONS FOR CONCRETE ROADS.





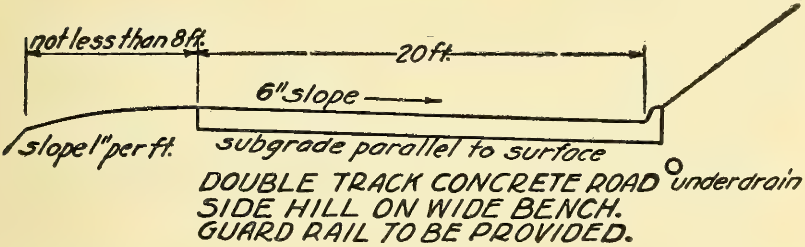


FIG. 1.—Continued.

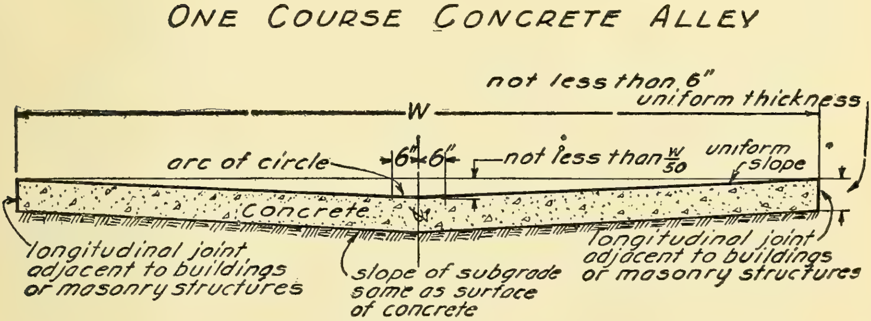

ONE COURSE CONCRETE ALLEY
not less than 6" uniform thickness
W
arc of circle
6"
6"
not less than W/50
uniform slope
Concrete
longitudinal joint adjacent to buildings or masonry structures
slope of subgrade same as surface of concrete
longitudinal joint adjacent to buildings or masonry structures

FIG. 2.—RECOMMENDED CROSS-SECTION FOR ONE-COURSE CONCRETE ALLEY.

although it is approached on a number of roads. It frequently happens that at a given time of day nearly all the traffic is in one direction, particularly the loaded vehicles, so that it is rare for two loaded vehicles to meet. If one vehicle must leave the paved portion of the road it will be the unloaded vehicle. These conditions, combined with the fact that there is so limited a mileage of improved roads in most sections of the country, together with the endeavor to build a road between certain points with half as much money as should be expended, have made it expedient, apparently, to construct many miles of single-track roads.

Experience has shown, however, that no sooner has a road been improved than the amount of traffic it carries is increased many fold. Scarcely a year passes before the community discovers that the road is too narrow. If it were possible to handle such matters in the most efficient way, it is doubtful whether there would be found public roads on which single-track construction could be justified.

Single-track roads, that is, those accommodating but one line of traffic, should be 10 ft. wide, with one edge laid on the center line of the right of way so that when converted into a two-track road the new portion may be added entirely at one side, bringing the completed two-track road in the center. For industrial pavements where pedestrian traffic is large, and not otherwise provided for, 12 ft. in width is recommended.

A two-track road should be not less than 18 ft. where a majority of the traffic is lighter vehicles, and 20 ft. where a considerable portion of the traffic is truck traffic.

A three-track road should be not less than 27 ft. and preferably 30 ft. if truck traffic develops to the extent of using two lines at a time.

A four-track road should be not less than 36 ft., preferably 40 ft., if a large portion of the traffic is truck traffic.

13. THICKNESS.

Two general classes of traffic, light and heavy, should be provided for with corresponding difference in thickness of the pavements. A light traffic road is one which carries, or is expected to carry, few or no vehicles over six tons gross weight. Such roads should be not less than 6 in. thick at the edges and not less than 8 in. thick at the center for 18- and 20-ft. roads.

For single-track roads thickness should be not less than six (6) inches. Where early widening to a double-track road is anticipated, it would be desirable to increase the thickness at the inside edge to not less than eight (8) inches, thus securing a flat sub-grade.

For heavy traffic roads, those roads carrying a considerable number of vehicles exceeding six tons gross weight, and not exceeding 15 tons, pavements should be not less than 8 in. thick at the sides and 10 in. at the center for 18- and 20-ft. widths. For other widths, the thickness should be as shown in adjoining table.

The following thicknesses with corresponding widths are recommended for light and heavy traffic roads respectively:

THICKNESS FOR LIGHT TRAFFIC ROADS AND STREETS.

WIDTH, FEET.	SIDES, INCHES.	CENTER, INCHES.
10	6 (outside)	8 (inside)
18	6	8
27	6	9
36	6	9½

THICKNESS FOR HEAVY TRAFFIC ROADS AND STREETS.

WIDTH, FEET.	SIDES, INCHES.	CENTER, INCHES.
20	8	10
30	8	11
40	8	12

14. CROWN.

The crown on concrete roads should be as flat as possible and need not exceed 1/100th of the width, except on single-track one-slope roads, which should have a crown of 1/50th of the width. Somewhat less crown can be employed on roads laid with a finishing machine than on those otherwise finished, owing to superior mechanical perfection of surface obtained.

On alley construction the crown should be the reverse of that for streets; that is, it should have a dished crown so that the water will run down the middle of the alley. It is recommended that the dished crown for alleys should be 1/50th of the width of the alley and that the subgrade for the alley be dished in the same manner so that the thickness of the pavement at every point will be the same. This should not be less than 6 in. for alleys in residential districts and not less than 8 in. for alleys that will be subjected to loads in excess of 6 tons.

15. SUPERELEVATION.

Curves should be superelevated to accomplish two purposes—to make them safe against skidding and to induce traffic to keep to the right side of the road and not tend to cut towards the inside as is the tendency on roads crowned in the ordinary manner around curves.

It is recommended that superelevation around curves should not exceed ¾ in. per foot. of width up to curves of 150 ft. radius; that curves 150- to 500-ft. radius should be given ½ in. per ft. superelevation; that on curves with a greater radius than 500 ft. the usual cross-section on level stretches should be followed around the curve.

It is recommended that embankments or other obstructions at curves should be so arranged as to afford around the curve a clear sight of not less than 500 ft.

In general, it is recommended that no highway be paved around a curve of less than 150 ft. radius at the inside edge of the pavement.

16. WIDENING TURNS.

The shoulder on the outer edge of curves, particularly where the radius is less than 150 ft., should be widened as much as possible and the ditch on the outer side of the road placed as far away from the pavement edge as the right of way will permit. On the inside edge the shoulder should not be less than 4 ft. wide.

It is recommended that on all curves of a radius less than 500 ft. the paved portion be widened as follows:

Radius feet.	Extra Width of Pavement on Inside of Curve, feet.
50	8
75	7
100	6
200	5
300	4
400	3
500	2

This added area on the inside of the curve is a "lune." In order that the lune may fit the curve properly and not appear "broken backed," the curve of the inner edge must be a true arc, tangent to the edge of the straightaway sections, and, therefore, it must start before the P. C. is reached. The lune may easily be staked out on the ground with transit and tape by means of data derived from the radius and central angle of the center line curve.

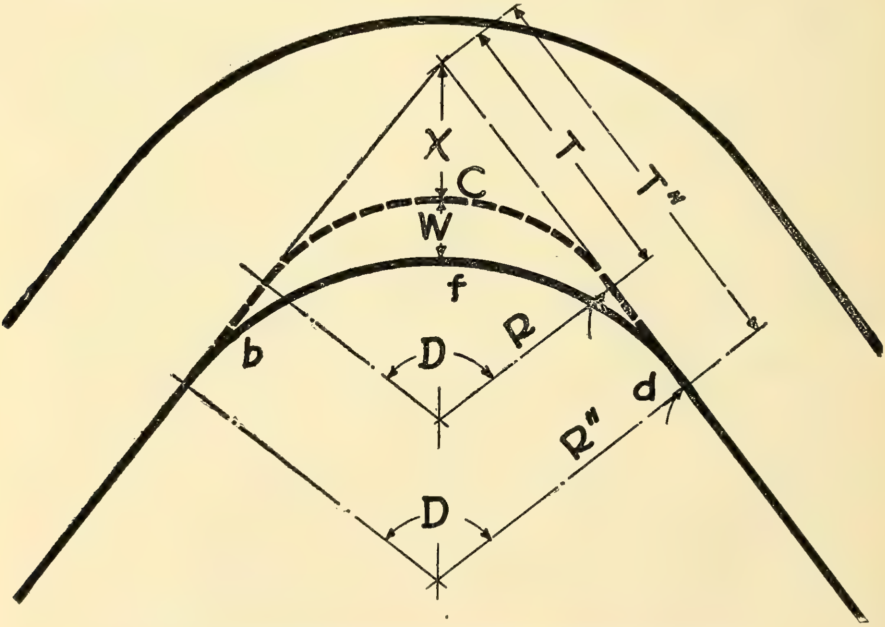


FIG. 3.—EXAGGERATED DIAGRAM SHOWING WIDENING ON TURNS.

In Fig. 3, R is the radius of inside edge of the pavement, if it is of uniform width around the curve. R'' is the radius of the inside edge when the curve is widened by W.

$$X = R \operatorname{exsec} \frac{D}{2}$$

$$R'' = \frac{X + W}{\operatorname{exsec} \frac{D}{2}}$$

$$T'' = R'' \tan \frac{D}{2}$$

The area of the lune *bcdf* in square yards =

$$\frac{\frac{\pi D}{360}\left[(R)^2 - (R'')^2\right] + (R + R'')\,(T'' - T)}{9}$$

17. TWO-COURSE CONSTRUCTION.

It sometimes happens that the aggregate that is available is not suitable for the surface of a concrete road so that suitable material would have to be shipped in, usually at a considerably higher cost. Under these circumstances it is advisable to build a two-course pavement; the top course to be made of the higher-priced and better-grade aggregate. It is recommended for two-course work, under these circumstances, that the lower course be made of a mixture of 1: 2½: 4, using the local aggregate, and that the top course, which is to be laid immediately after the lower course has been spread and rolled, but not belt finished, be 2 in. in thickness and consist of a mixture of 1: 1½: 2½, using an aggregate not to exceed 1 in. in size.

18. INTEGRAL CURB FOR STREETS.

A concrete pavement and curb can be constructed at the same operation, the whole forming a unit. Such a curb is known as "integral curb" because it is a part of the pavement itself. It is suitable for boulevard, street, avenue, park drive, business thoroughfare, alley, driveway or road.

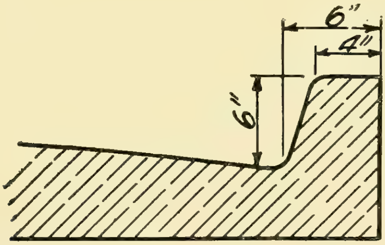
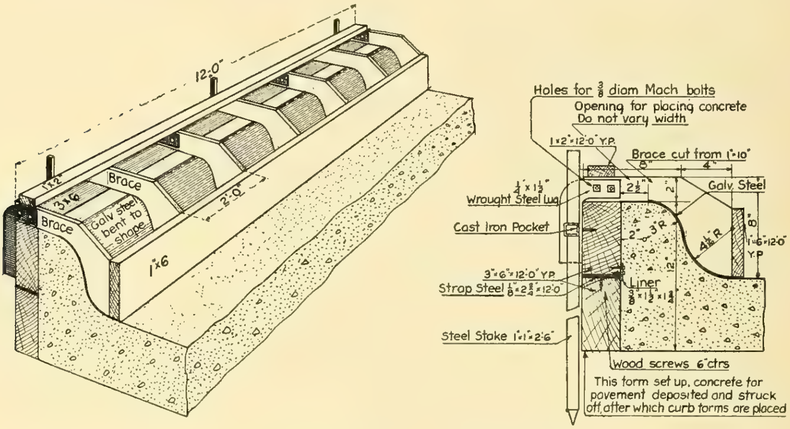


FIG. 4.—INTEGRAL CURB FOR CONCRETE ROAD.

The form of curb which is recommended is one having a curved batter. It allows the full width of the street to be used, as it encourages drivers of cars to stop their machines close to the curb, there being no possibility that the tires or rims can be damaged. The natural tendency is for the driver to stay clear of the curb having a vertical face, by as much as 1 to 1½ ft., so as to avoid injury to the wheels of his machine. If cars on both sides of the street stop in this manner the effective width of the street may be reduced by as much as 3 ft. It must also be remembered that heavy wagons with steel tires sometimes back against the curb, sharp corners will be chipped off and the curb soon becomes unsightly. With the curb having a curved batter, however, there is no sharp corner and tires striking against it find no place where a piece can be easily chipped out.



DETAIL OF CURB SECTION AND FORM

FIG. 5.—FORM FOR INTERGAL CURB CONSTRUCTION.

The integral curb possesses features of distinct sanitary value when made with rounded corners. A curb which makes a sharp angle at its intersection with the pavement affords lodgment for dust and dirt. The sharp angle is difficult to clean. The curb with the rounded intersection offers no such collecting place and is much easier to keep free from rubbish. In fact, it is practically self-cleaning as the abrasive action of motor tires and wheels of other vehicles are effective in breaking up the deposits here, so that the first rain or flushing of the street will insure a clean gutter.

The construction of the integral curb has been made extremely simple by the development of efficient forms, one of which is shown in Fig. 5. There are two methods of using this, each having its advantages. Some contractors have preferred to put their forms into position and fill them with the concrete at the same time that the pavement is being placed. The advantage here is that there is no necessity for handling a separate part of the mix for the curb, enough concrete being deposited along the curb forms to fill them at the same time that the concrete is being placed for the pavement proper. The forms are filled from the top and concrete thoroughly spaded to insure uniformity.

When the concrete has hardened sufficiently the forms are removed and any stone pockets appearing on the surface are filled and the curb troweled over with a wooden float.

To secure a final finish that is smooth and uniform the curb is gone over with a whitewash or calcimine brush which has been dipped into a bucket of water and shaken out. The brush is used with a horizontal stroke and gives a perfect finish to the curb.

With the second method of building the integral curb it is placed after the pavement has been nearly finished. For this purpose it is necessary that the back form be in two parts; the upper part holding the curb mold is not placed until the surface of the pavement has been rolled as many times as necessary to remove the excess water and given the first belting. Then the upper half of the back form is placed and the mold for the curb attached and filled with concrete which is thoroughly spaded.

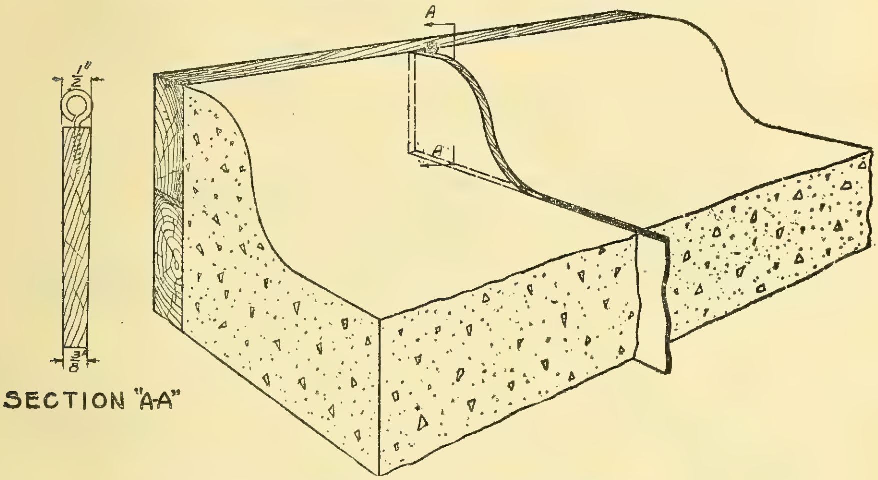


FIG. 6.—SEPARATOR FOR INTEGRAL CURB.

This method possesses the distinct advantage that the surplus water removed from the concrete in finishing will run off along the edge of the pavement of its own accord, whereas if the curb is placed with the pavement itself the excess water will flow to the gutter and must be removed. This is usually accomplished by using an ordinary street broom to brush the water along the gutter and off onto the subgrade.

It should also be noted that where a slight grade of several blocks is encountered it is well to build the road or pavement down grade so that the excess water removed in finishing will flow off onto the subgrade instead of back over the finished work.

A ½-in. joint in the curb should be made at every joint in the pavement and should be in perfect alignment both transversely and vertically with the joint material.

Where the curbs are molded to shape by the use of forms already described, the joint in the curb should be made with a tapered separator of oiled wood ½ in. thick at the top and cut to the exact section of the curb. This is held rigidly in place by a long hairpin while the material is being deposited in the forms. After the concrete is sufficiently hardened the curb forms are removed and the curb finished. The separator is tapped lightly until loosened and then withdrawn about ½ in. The edges of the curb at the joint should be touched up with a pointing trowel to break the sharpness. The separator is then removed, care being taken not to break the side walls of the curb, and the joint is left open, no filler being used. (See Fig. 6.)

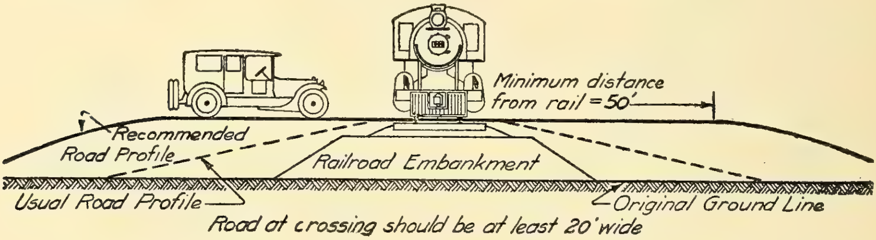


FIG. 7.—RECOMMENDED PROFILE FOR RAILROAD CROSSINGS.

CHAPTER III.

GRADING AND DRAINAGE.

19. GRADING.

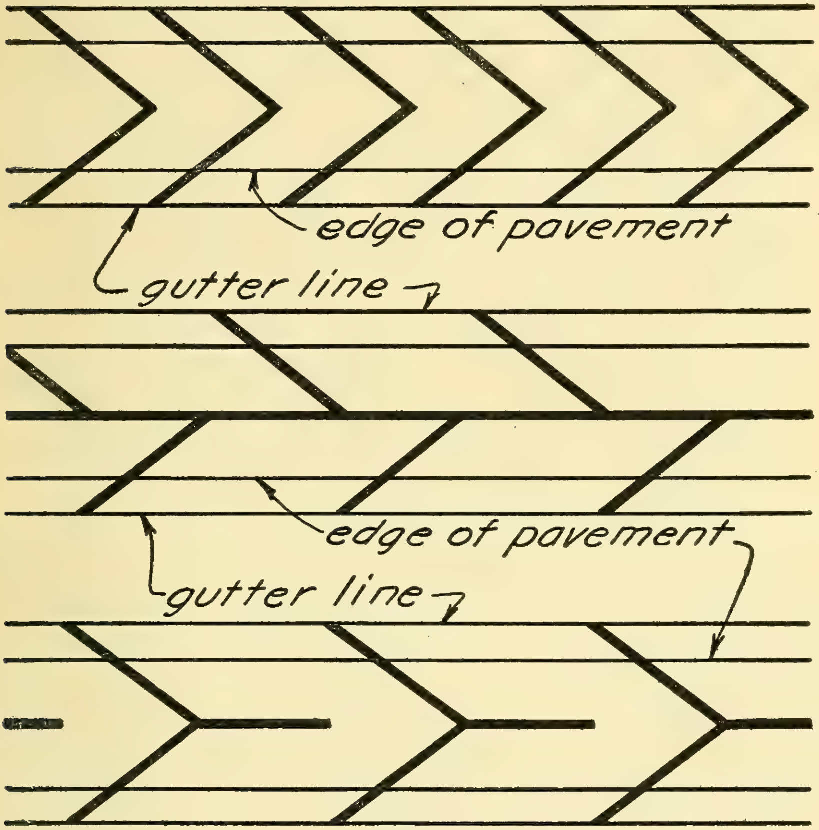
In general, the requirements for grading of concrete roads are not essentially different from those pertaining to roads of other types of construction, so that there is little to be suggested for recommended practice that is essentially peculiar to concrete roads.

One point may well be emphasized, which is the profile to be established at railroad crossings. When it is not practicable to effect a separation of grades, grade crossings can be made much safer than otherwise if care is taken to have the road surface for a distance of at least 50 ft. on either side of the railroad track at the same elevation as the top of the rails. (Fig. 7.)

Similarly on approaches to bridges it is desirable that the grade of the road, for at least 50 ft. at each side of the bridge, shall be at the same elevation as the bridge floor.

20. DRAINAGE.

The general features of road drainage are divided between surface and subdrainage. The requisites for surface drainage are a proper crown, slope of the shoulders to the gutters, and outlets for the water from the gutters through culverts and small bridges. Subdrainage is effected usually by laying a tile drain and backfilling the trench in which the drain is laid with rock, gravel or other porous material.



V DRAINS

The steeper the grade the greater the angle of the V.

FIG. 8.—INSTALLATION OF DRAINS.

The essential point to keep in mind in draining a concrete road is to secure as nearly a uniform condition of subgrade as possible. This may be either a wet or dry condition. Thus over swampy ground it is practical to lay a concrete road without placing as high a fill as is necessary with macadam types of construction in order to secure for the macadam a dry roadbed.

Under these conditions, especially if the ground is very unstable, extra reinforcement should be used, placed near the bottom of the slab. Such reinforcement may consist of half-inch bars running both transversely and longitudinally, spaced 18 in. on center; the concrete slab should be not less than 8 in. in thickness.

A frequent source of considerable trouble may be pointed out. It is not uncommon on hills to find the character of the soil at the top considerably more porous and open than part way down the hill. And it will be noticed, wherever the more porous soil joins the closer textured soil there will be an accumulation of water flowing along this seam. Where this seam intersects the roadbed there will, in wet seasons, always be found a seepy place. It will be necessary in such places, if trouble is to be avoided, that underdrainage be provided, which will reach into the porous soil sufficiently to intercept the water which it carries so it will not be deflected towards the surface of the road along the juncture of the more open with the denser soil. Unless there are indications which show clearly that in such situations the water is coming from one side of the road, the drain should extend under the center of the road from the porous soil and then split in two parts and discharge at each side further down the hill, running as a subdrain just under each edge of the pavement after the clay soil is reached.

Particularly in flat country, shoulder drains should always be provided when the concrete is laid on clay subsoil. These shoulder drains are cut through the shoulder for the width of a shovel from the subgrade to the bottom of the gutter. They should be cut at an angle of about 45° in the direction of the road and filled with broken stone to within 3 in. of the surface and then covered with material of which the shoulders are made. Shoulder drains should be put in at intervals of 50 ft. on each side of the road.

On sections of the road where deep cuts have been made through clay soil, cross ditches 18 in. deep and about 1 ft. in width, should be placed under the surface of the road, starting at the center line, and thence at an angle of about 45° to the flow line of each gutter, the ditches to be filled with crushed stone or gravel. These ditches make a V with the point of the V at the center line, the angle of the V depending upon the slope of the hill. They should be put in at intervals of 75 to 100 ft. The first one should be just above the cut and the last one at the end of the cut.

CHAPTER IV.

ORGANIZATION AND EQUIPMENT.

21. TWO-BAG BATCH WORK.

The organization of the concreting crew is dependent in a large measure upon the varying conditions that are met with on every job.

A two-sack batch mixture is a convenient and economical size to operate for most pavement work, the width of which is not more than 20 ft. Many contractors prefer the three-sack mixer which permits greater yardage per day and usually assures a longer time in the mixer. The following diagram shows the division of labor for a crew for average standard equipment.

It will be noted that on some jobs some of these men can be eliminated. For instance, subgrading under the boom is not needed if proper attention has been given to the subgrade when first constructed.

The equipment needed for two-bag batch operation is as follows:

700 lin. ft. forms.
18 shovels.
8 wheelbarrows.
1 2-bag batch mixer.
1 roller with handle or ropes
1 1-in. radius edger.
2 belts.
250 ft. hose.
2 wooden floats.
1 strikeboard.
1 cross-section header board.
1 heavy wire cutter for reinforcing.
2 small hand nippers for opening sacks.
water pipe.
stakes.
sledge.

ORGANIZATION OF CREW FOR CONCRETE PAVEMENT CONSTRUCTION—TWO-BAG BATCH.

Based on widths up to 20 ft.; one-course, 1:2:3; boom and bucket delivery.

Contractor or Superintendent	Foreman (1)	Forms	Form Setter	(1)	
			Helper	(1)	
		Mixing and Placing	Shovelers and Wheelers	(5)	Circulating system, 4 wheelers; 1 shoveler.
			Cement Handlers	(3)	1 handling cement; 1 handling bags; 1 bundling sacks and handling.
			Engineer of Mixer	(1)	
			Fireman	(1)	Also sprinkles subgrade.
			Batch Operator	1)	Not needed for standard equipment.
			Subgrader	(1)	For subgrading under boom, unnecessary on some jobs.
			Concrete Distributors	(2)	
			Water Boy	(1)	
			Reinforcing and Joint Installers	(2)	
		Finishing	Strikeboard operators Tampers Rollers and Belters	(4)	2 on strikeboards, 2 on roller and belts.
		Curing and Protection	Shovelers	(2)	2 shovelers cover as much of pavement as has hardened sufficiently, leaving what is too green for gang to cover next morning; also keep earth covering of pavement wet. Entire crew covers remainder of pavement each morning before beginning concreting.

ORGANIZATION OF CREW FOR CONCRETE PAVEMENT CONSTRUCTION—THREE-BAG BATCH.

Based on width up to 20 ft.; one-course, 1:2:3; boom and bucket delivery.

Contractor or Superintendent	**Foreman** (1)	**Forms**	Form Setter	(1)	
			Helper	(1)	
		Mixing and Placing	Shovelers and Wheelers	(10)	1 gang, 5 shovelers; 1 gang, 5 wheelers; wheelers and shovelers may alternate or not.
			Cement handlers	(3)	1 handling cement, 1 handling bags from piles, 1 bundling sacks and handling.
			Engineer of Mixer	(1)	
			Fireman	(1)	Also sprinkles subgrade.
			Batch Operator	(1)	
			Subgrader	(1)	For subgrading under boom; unnecessary on some jobs.
			Concrete Distributors	(2)	
			Water Boy	(1)	
			Reinforcing and Joint Installers	(2)	
		Finshing	Strikeboard Operators Tampers Rollers and Belters	(4)	2 on strikeboards, 2 on roller and belts.
		Curing and Protection	Shovelers	(2)	2 shovelers cover as much of pavement as has hardened sufficiently, leaving what is too green for gang to cover next morning; also keep earth covering of pavement wet. Entire crew covers remainder of pavement each morning before beginning concreting.

ORGANIZATION OF CREW FOR CONCRETE PAVEMENT CONSTRUCTION—THREE-BAG BATCH.

Based on width of 20 to 40 ft.; two-course, bottom 1:2½:4; integral curb; boom and bucket delivery.

Contractor or Superintendent	**Foreman** (1)	**Forms**	Form Setter	(1)	
			Helper	(1)	
		Mixing and Placing	Shovelers and Wheelers	(14)	Based on 2 gangs one up and one loading
			Cement Handlers	(4)	1 handling cement in skip, 2 handling bags from wagons and piles, 1 bundling sacks.
			Engineer of Mixer	(1)	
			Fireman	(1)	Also sprinkles subgrade.
			Batch Operator	(1)	
			Subgrader	(1)	For subgrading under boom, unnecessary on some jobs.
			Concrete Distributors	(2)	3 needed on 40 ft. streets.
			Reinforcing and Joint Installers	(2)	
			Water Boy	(1)	
		Finishing	Working Foreman	(1)	
			Luter	(1)	Also rolls and belts.
			Finishers' Helpers	(2)	Assist with curb work and rolling and belting.
		Curing and Protection	Shovelers	(2)	2 shovelers cover as much of pavement as has hardened sufficiently, leaving what is too green for gang to cover next morning; also keep earth covering of pavement wet. Entire crew covers remainder of pavement each morning before beginning concreting.

22. THREE-BAG BATCH WORK.

The typical organization of a crew for pavement work where the width is from 20 to 40 ft., two-course construction with integral curb based on a mix of 1 : 2½ : 4 for the first course, using standard equipment and boom and bucket delivery is as follows:

The equipment needed for a three-bag batch operation for work up to 20 ft. is as follows:

1300 lin. ft. forms.
20 shovels.
12 wheelbarrows.
1 3-bag mixer.
1 roller.
1 1-in. radius edger.
2 belts.
1 heavy wire cutter for reinforcing.
2 small hand nippers for opening sacks.
1 cross-section header board or joint installer.
water pipe.
350 feet hose.
stakes.
sledge.

The equipment needed for a three-bag batch operation for pavement construction 20 to 40 ft. wide is as follows:

500 lin. ft. forms.
300 lin. ft. curb forms.
24 shovels.
18 wheelbarrows.
1 batch mixer.
1 heavy wire cutter for reinforcing.
2 small hand nippers for opening sacks.
1 split float.
1 split roller with ropes.
1 to 3 metal hand trowels.
1 cross-section header board or joint installer.
1 bridge.
1 lute.
1 set of 3 T's or stakes for sighting.
2 wooden hand floats.
1 long-handled float.
2 belts.
350 ft. hose.
water pipe.
stakes.
sledge.

CHAPTER V.

HANDLING AND HAULING MATERIALS.

23. UNLOADING YARD.

Sufficient space is needed for bins, stock piles and cement storehouse in unloading yards where materials are shipped in by rail. With uncertain transportation facilities, storing material at the siding is very desirable.

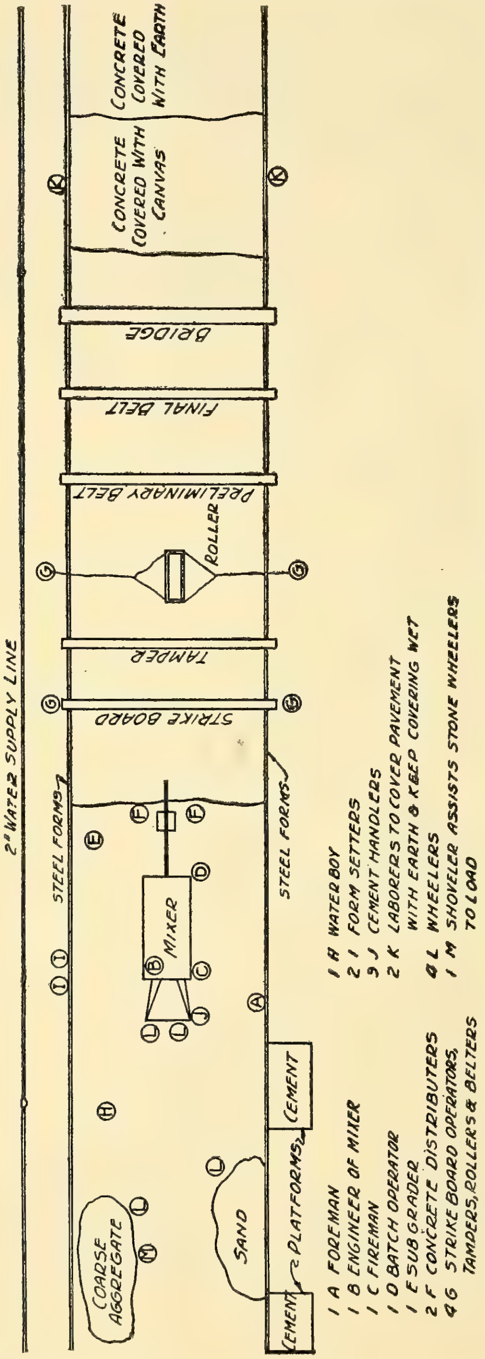


FIG. 9.—ORGANIZATION FOR 1-COURSE PAVEMENT CONSTRUCTION, USING 2-BAG MIXER.

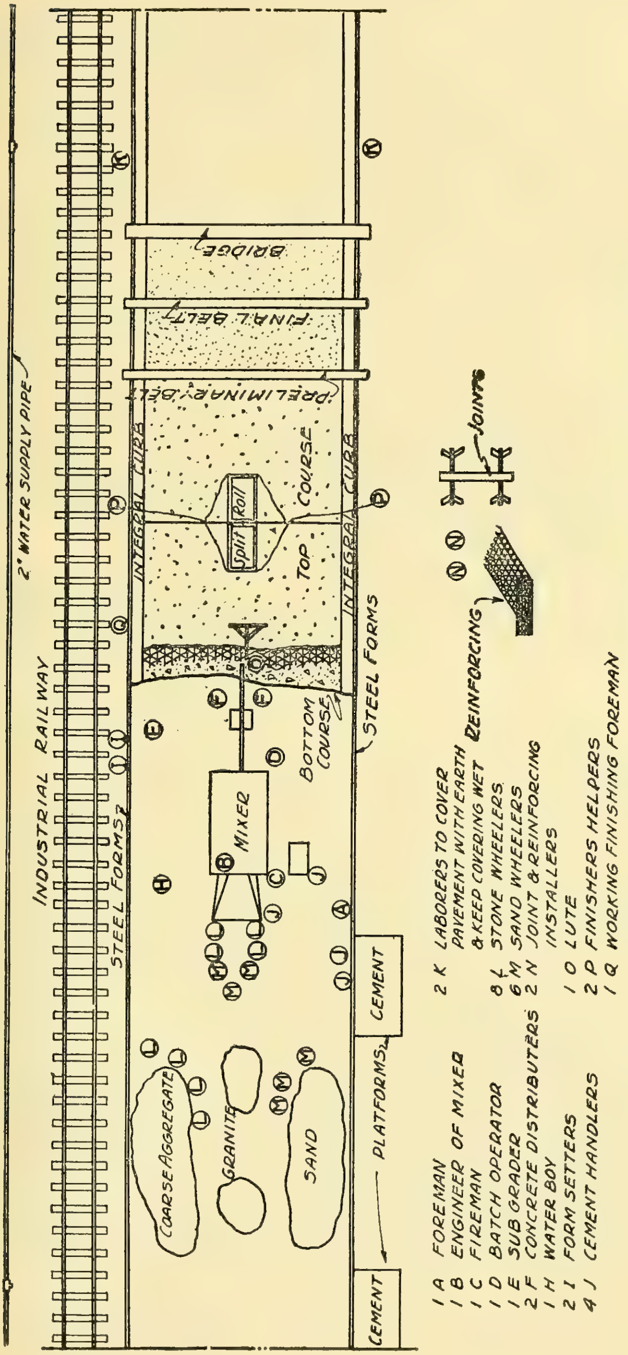


FIG. 10.—ORGANIZATION FOR 2-COURSE PAVEMENT CONSTRUCTION, USING 3-BAG MIXER.

If an industrial railway is used for hauling, a more extensive yard is necessary than with teams or truck hauling. For profitable operation an industrial railway should be in use constantly. A crane, derrick with clam-shell bucket, bucket conveyor, or other power unloading device should be provided. Equipment that can be easily arranged about the yard is much preferred to stationary apparatus.

24. HAULING EQUIPMENT.

Horse-drawn dump wagons, motor trucks, tractors and industrial railways are all used for hauling materials. For short distances over bad roads wagons may be most economical.

Trucks are coming into wide use and, where the roads over which they operate are in fairly good condition, are to be preferred to team hauling. Three-ton capacity trucks, capable of good speed, are usually more efficient than the larger size when used over the average unimproved road or pavement subgrade. The smaller trucks do not cut up the subgrade so badly and are easier to turn in cramped quarters than the larger machines. Industrial railways are adapted for use where the job is large, the average haul several miles, and unloading facilities ample. The investment is high and the equipment, therefore, must be kept in constant use for economic operation. Hauling is largely independent of weather conditions, but grades limit the speed and capacity of trains where slopes steeper than 4 per cent are encountered. Geared locomotives are ordinarily used where grades exceed 4 per cent; a locomotive of this type is much slower than a direct-acting locomotive. One great advantage of the industrial railway equipment lies in the fact that the subgrade is not cut up by hauling over it.

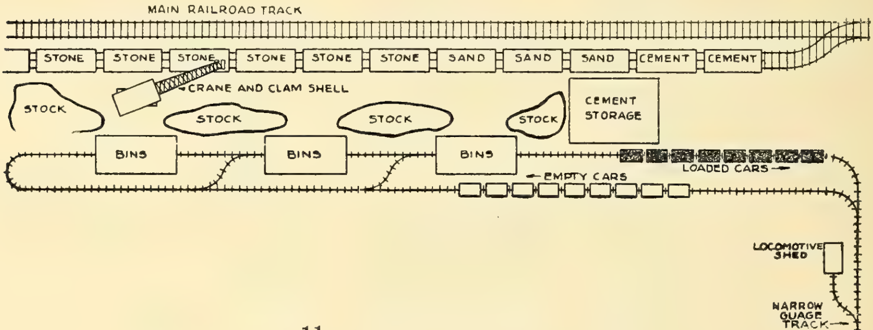


FIG. 11.—PLAN OF UNLOADING YARD.

Materials may be dumped direct into the mixer from cars when the hauling is done over industrial railway. This method is recommended because labor is saved, larger mixers may be used, aggregates are kept clean, and material is not wasted on the subgrade. Where this plan is followed it is absolutely necessary that a continuous supply of materials be delivered to the mixer; sufficient equipment under proper supervision should be provided to make that certain. Good track construction and maintenance are of great importance.

One and one-half yard dump cars of 24-in. gauge are commonly made use of on industrial railways. The light 32-in. steel ties frequently found on built-up track sections are unsatisfactory, as longer ties are needed. Wooden ties have been used successfully, but steel ties in built-up sections are more convenient than separate ties and rails.

25. PLACING ON SUBGRADE.

The fine and the coarse aggregates should be piled on opposite sides of the subgrade if possible and kept separate. Care should be taken to place aggregates on the subgrade in the quantities needed so that unnecessary labor and time will not be required to wheel the materials long distances to the mixer. Ample storage on the road is desirable unless the aggregates are dumped directly into the mixer.

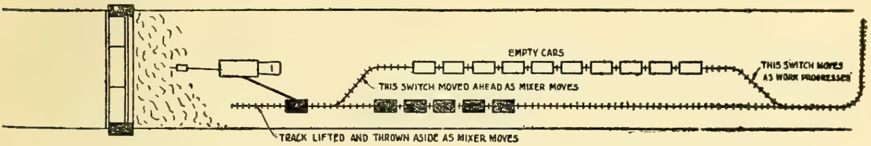

EMPTY CARS
THIS SWITCH MOVES AS WORK PROGRESSES
THIS SWITCH MOVED AHEAD AS MIXER MOVES
1
TRACK LIFTED AND THROWN ASIDE AS MIXER MOVES

FIG. 12.—TRACK ARRANGEMENT AND PLAN OF CONCRETE PLANT WHEN TRACK IS ON SUBGRADE.

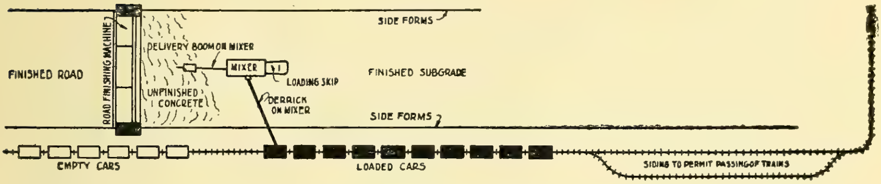


FIG. 13.—TRACK ARRANGEMENT AND PLAN OF CONCRETE PLANT WHEN TRACK IS ON SHOULDER.

Cement should be piled on boards or racks and shelter provided for it in case of rain.

Coarse aggregate when delivered upon the subgrade by either motor truck or teams very often requires a re-gradation before being placed in the "skip" ready for hoisting. Primarily the need for this is due to the number of times the material is handled before it reaches the job, thus separating the smaller sizes from the larger. The readjustment to a better graded condition may be made by the manner in which the wheelbarrows are loaded. The attention of the inspector is required to get proper results when the necessity demands this practice. This will prevent an uneven stony surface which will not roll nor belt with satisfactory results.

26. WATER-SUPPLY.

An ample supply of suitable water is necessary for concrete pavement work. Possible sources are city mains, wells, springs, tile drains, streams and lakes. If careful investigation of all available sources of water supply near the work does not reveal a sufficient quantity to insure plenty of water regardless of the weather, it may be economical to sink 6-in. wells. In some instances small reservoirs or standpipes have been constructed to furnish small storage and to equalize pressures.

The water supply system should be of such a type as to require little attention and should furnish the necessary amount of water at all times at as uniform a pressure as possible. A contractor in making bids in the spring, when streams are high, should make sure by inquiry among local residents as to the dry season flow before placing dependence on any stream for his water supply.

The size and type of the pumping plant depend upon the length and grade of pipe line and the size and condition of the pipe. Where a two-sack mix is used, the pipe line should be able to deliver at least 25 gal. of water per minute at a pressure of approximately 50 lb. per sq. in. at any point on the pipe line. With a four-sack mix, the line should be able to furnish at least 40 gal. per min. A triple plunger pump, either belt or gear connected to a gasoline engine, has been used with success in such work. The small additional sum required for the purchase of a good pump is well spent.

If water is to be taken from a deep well, it will be necessary to use two pumps and a storage tank or reservoir—a deep well pump to raise the water from the well to the reservoir and a pressure pump to force it through the pipe line from the reservoir. In order to avoid possible damage to the pumping system, due to water hammer caused by shutting off valves quickly and in order not to overload the pump, it is necessary to place a relief valve in the line just ahead of the pump, with a bypass discharging into the source of supply. This valve should be set to open when the pump pressure exceeds that needed. When the valve opens, it will discharge the water back into the source of supply without waste. The diameter of the pipe line should be not less than two inches.

In addition to the loss of head, due to friction, account must be taken of the grade of the pipe line, since sufficient pressure must be provided for at the pump to give the required amount of water at the highest point of the line. The pipe line capacity may be greatly reduced by the rusting of the pipe, by formation of deposits, or by inserting lengths of smaller pipe.

T's for supplying water to the mixer and for sprinkling should be placed in the pipe line at intervals of about 200 ft. It is advisable to insert at least five gate valves per mile in the pipe line and enough additional valves should be provided so that one may be placed on each T for a sufficient distance back of the mixer to allow sprinkling for the required length of time. Hose not less than 1½ in. in diameter should connect the pipe line with the mixer in order to insure the filling of the mixer tank while a batch is being mixed and dumped. In laying the pipe line, drain valves should be placed at all the low points in order that when not in use the water may be drained out and thus avoid splitting of the pipes due to freezing. Neglect of this detail has caused great losses to concrete road contractors.

The amount of water needed for curing alone is estimated at 25-30 gal. per sq. yd. Even greater amounts may be needed in hot summer weather. For mixing, not over 8 gal. per sq. yd. (average thickness of pavement 7 1/3 in.) should be needed and a smaller quantity should be used unless aggregates are very dry. In sprinkling the subgrade the amount required will vary, but an allowance of 1½ gal. per sq. yd. for sand or ½ gal. per sq. yd. for clay, should be used in figuring the quantity of water needed.

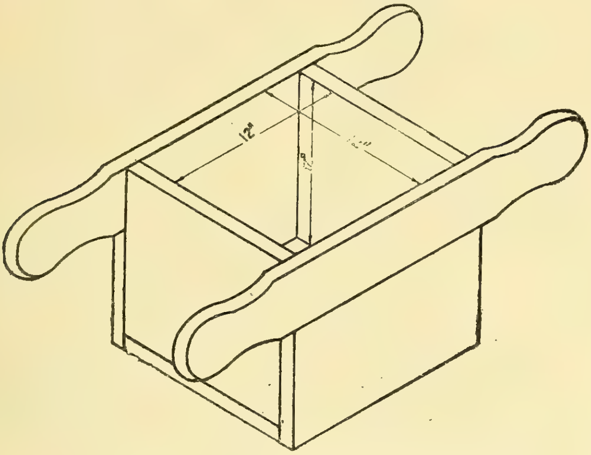


FIG. 14.—ONE CUBIC FOOT MEASURING BOX.

CHAPTER VI.

MIXING AND PLACING CONCRETE.

27. TIME OF MIXING.

Materials should be mixed for a full minute. In general it is to be recommended that mixers should be supplied with timing devices so as to insure that each batch will get the proper amount of mixing.

28. MEASURING MATERIALS.

It is recommended that wheelbarrows holding either 2 or 3 cu. ft. be used. A 1-cu.-ft. measuring box should be provided (Fig. 14) and capacity of all wheelbarrows checked before they are used.

Where the size or proportions of the batch do not permit the use of known capacity loads in the barrows, a paint mark should be inscribed on the inside of the pan of the barrow at a height as checked by the measured quantity of material required, determined with a measuring box when leveled.

29. SIDE FORMS.

Side forms which control the placing of concrete must be held rigidly in place. It is recommended that the side forms be of steel and if the amount of work extends over a mile or two, they will be found more economical than wood. Whatever the type of forms, it is recommended that stakes be driven upon which the forms are to rest; the tops of the stakes being at such an elevation that when the form is placed upon them the top of the form will be at the required elevation. For this purpose stakes of wood 2 × 2 × 12 in. in length are to be used. A stake should be placed near the end of each section of the form so as to insure true alignment where one section joins another and intermediate stakes placed at intervals of 4 or 5 ft. In addition to these stakes other stakes must be set sufficient to hold the forms to line. The joints in the side forms should not come opposite each other, but should be staggered. (Fig. 16.)

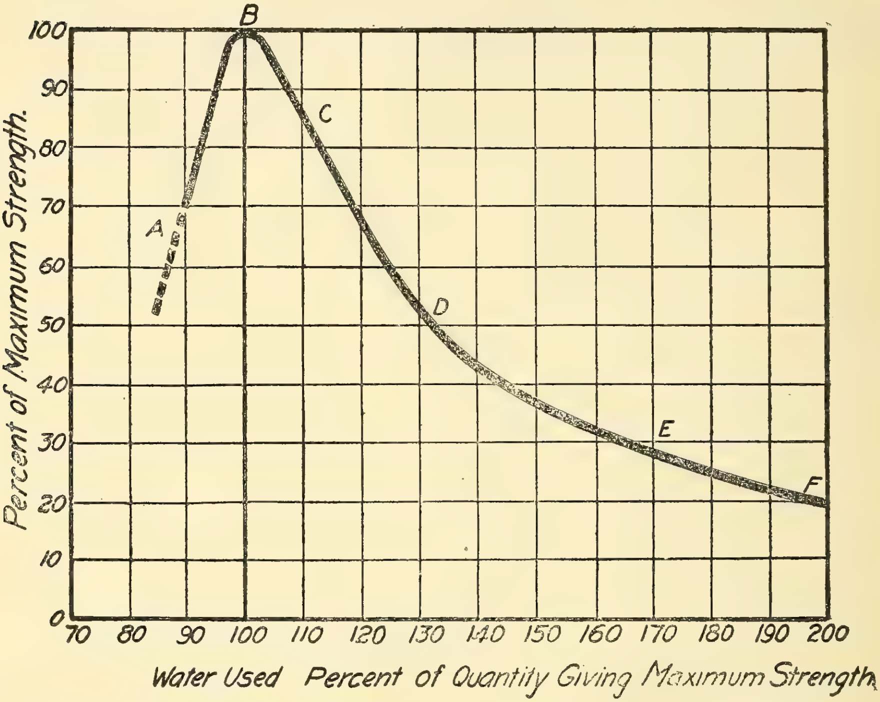


FIG. 15.—CURVE SHOWING EFFECT OF WATER USED IN MIXING ON STRENGTH OF CONCRETE. POINT B REPRESENTS MAXIMUM STRENGTH OBTAINABLE FROM MIX. C TO D IS PROPER CONSISTENCY FOR ROAD WORK. FROM D TO F STRENGTH DECREASES RAPIDLY AS WATER IS INCREASED.

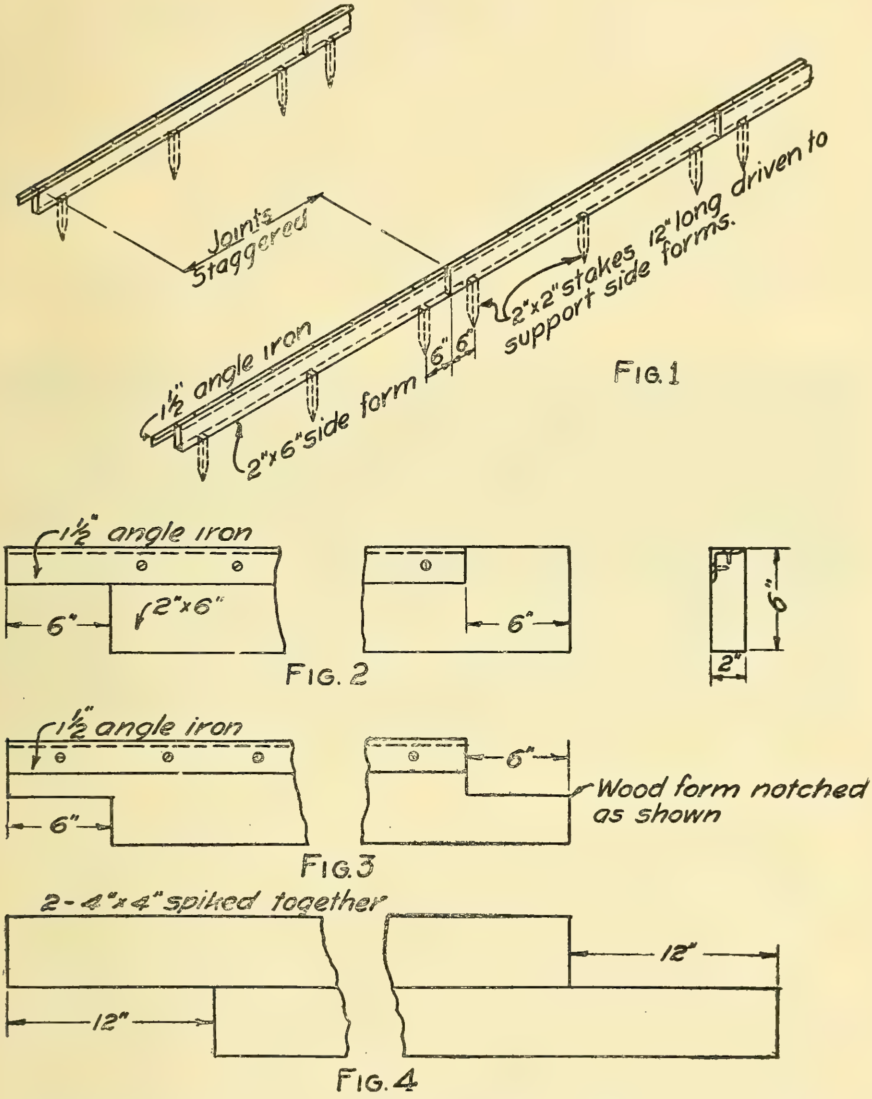


FIG. 16.—FORMS SHOULD BE PLACED ON STAKES AS SHOWN IN FIG. 1 ABOVE. FIGS. 2, 3, 4, ABOVE SHOW FORM CONSTRUCTION.

30. PLACING CONCRETE ON SUBGRADE.

The necessity for care in placing concrete from the mixer onto the subgrade has come to be more fully appreciated as more experience has been gained. If it happens for any cause that a batch of concrete as deposited in the road tends to have portions of the aggregate segregated it is of importance that the concrete shall be redistributed so that there shall be no segregation, particularly towards the surface. The drier the mixture the less tendency there is for this segregation, which furnishes an additional reason for care in the amount of water used in the mixer.

The subgrade should be firm and of as uniform density as possible. If it is dry or dusty it may be lightly sprinkled, but it should not be made wet. Concrete may be deposited from the mixer from an open chute or boom and bucket. If buckets are used which discharge by the opening of doors at the bottom, it is necessary to watch them carefully that no stone lodge in them to prevent their closing tightly. If the chute method is used the pitch of a chute should be steep enough to deliver concrete of a proper consistency. A chute of less angle may be used if a man with a shovel is kept at hand always to move the concrete along the chute.

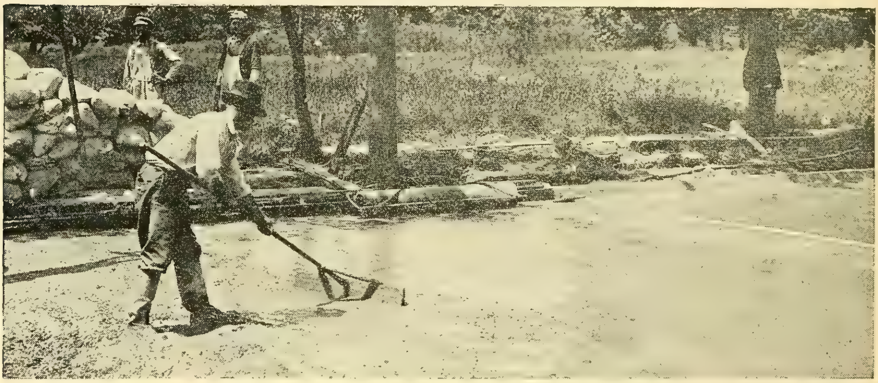


FIG. 17.—USE OF LUTE TO PLACE CONCRETE.

On roads and streets not over 20 ft. in width, it is practical to use a template cut to the proper crown and thus distribute the concrete over the entire pavement. If the strikeboard is made of wood it should be shod on the striking edge with a strip of steel. It is recommended that two strikeboards be used. The first one should have a 3- or 4-in. face, which is used as a tamper as well as to strike the concrete. (Fig. 24.) This board is to be cut so as to leave about ½ in. surplus concrete at the center. This strikeboard should be followed by a lighter board which is moved continuously over the surface without the tamping motion, taking off the slight surplus of concrete left by the tamping board.

On pavements wider than 20 ft. it is recommended that concrete be distributed by luting. (See Figs. 17-18.) In this method a lute which resembles a toothless rake replaces the strikeboard. The lute is operated so as to place the concrete approximately to the finished section. This is

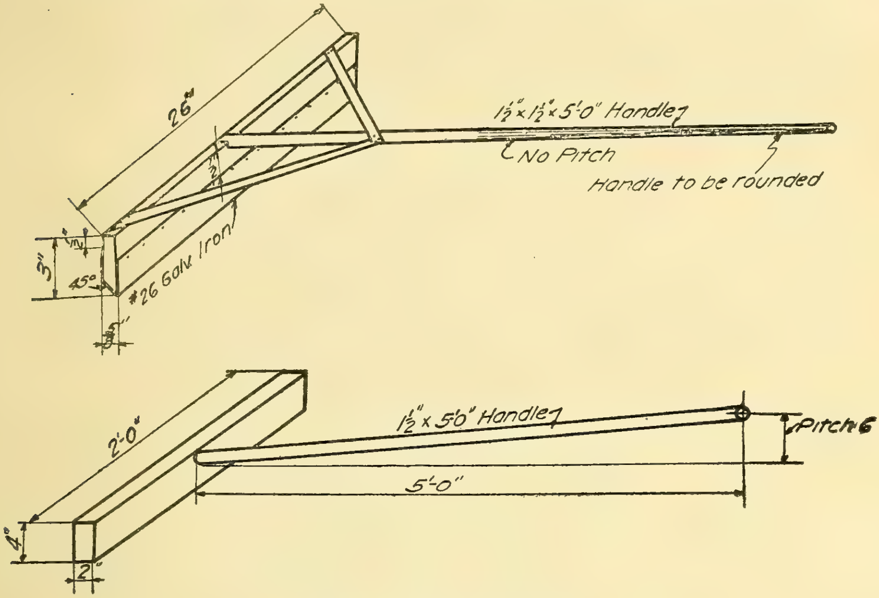


FIG. 18.—TYPES OF LUTES.

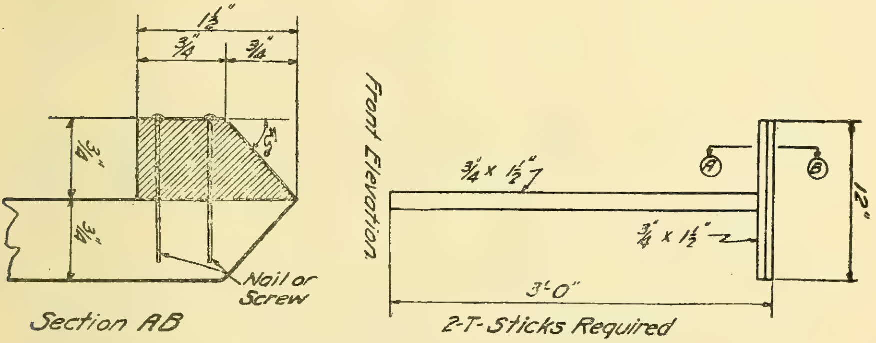


FIG. 19.—T-STICK FOR SIGHTING IN POINTS.

accomplished by placing steel pins across the road, the number depending upon the width; not less than three should be used and usually the number need not exceed eight. One pin should be placed on the center line and at least one on the quarter points. Rows of pins are placed across the road at intervals of 7 to 8 ft. These pins are set with their tops 2 in. higher than the surface of the concrete so that they may be readily seen and pulled out when the concrete has been placed around them. The pins are readily set in place by the use of T's. A sight stick, on which the elevations for different points of the cross-section are marked by notches, is used to determine the elevation of each pin. The slight unevenness that is left

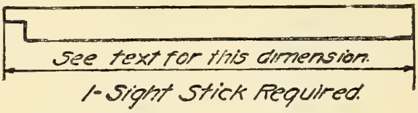


FIG. 20.—DETAIL OF SIGHT STICK.

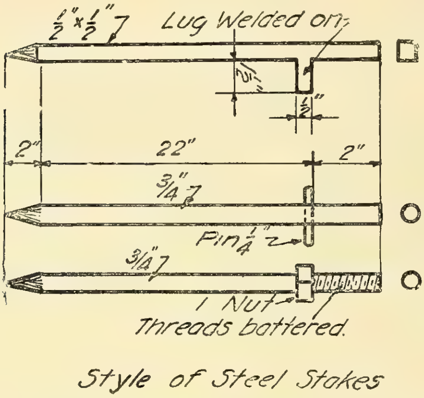


FIG. 21.—STEEL STAKES USED TO MARK POINTS FOR LUTING.

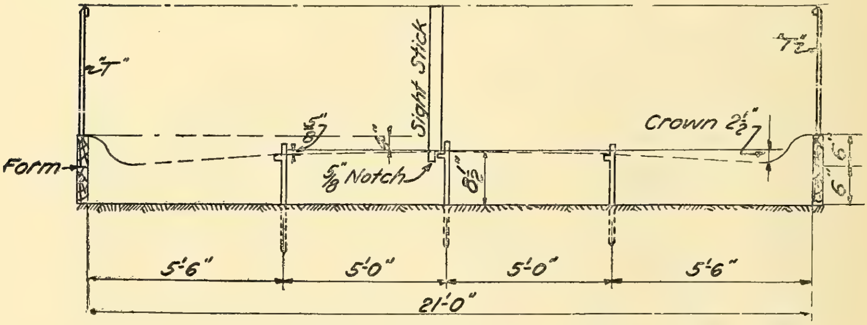


FIG. 22.—METHOD OF SIGHTING IN POINTS WHEN LUTING METHOD IS USED TO OBTAIN PAVEMENT CONTOUR.

in the surface of the pavement after it is luted is entirely removed by the use of the roller and belt described under "Finishing."

31. PLACING IN COLD WEATHER.

The construction of concrete pavements during freezing weather is not recommended because of hazards and the added cost made necessary by the protection of the concrete.

It is sometimes necessary to finish up a job of concrete street or road paving which is nearly completed during a time when the temperature may drop below freezing and on such work caution should be taken to avoid any possibility of the concrete being damaged by frost.

Concrete should not be placed on a frozen subgrade. Methods to be followed in combating the injurious effects of cold weather may be grouped under the following headings:

(A) Application of artificial heat to materials or finished work, and means of preventing the dissipation of heat developed during the setting and hardening of the concrete. (Heating of water or aggregates; heating fresh concrete by steam lines; the use of covering, etc.).

(B) Use of chemical compounds which will lower freezing point of mixing water, or accelerate the setting and hardening of the cement.

Special care must be taken to avoid the use of lumps of frozen aggregate. On account of the frequent moving of equipment, heating of the aggregates is not always practicable in road work although this can be done in other kinds of concrete construction.

If a road roller or other source of steam supply is available, the aggregate piles may be heated by inserting steam pipes carrying live steam. This heating will be greatly facilitated by covering the piles with canvas.

Valuable results can be obtained at little expense by heating the mixing water. This can best be accomplished by inserting an improvised steam coil in the water line near the mixer. The water coil may be heated by a wood or coal fire. It is not necessary that the water be extremely hot, as it will accomplish a usful purpose if this method serves to remove only the chill from the aggregate and impart some heat to the fresh concrete. Satisfactory heaters for this purpose are on the market.

The chemical reactions which accompany the setting and hardening of the concrete generate some heat, and where there are indications that the temperature will fall below freezing during the few hours following the placing of the concrete, it is important that this heat be retained. This may be accomplished by placing a covering of canvas or other material over the newly-laid road in such a manner as to leave an air space above the concrete. Disturbance of the enclosed air should be prevented as much as possible by weighting the canvas with timbers, sand or earth.

In the event of an extreme drop in temperature, pipe lines supplied with live steam may be carried under the canvas to protect the fresh con-

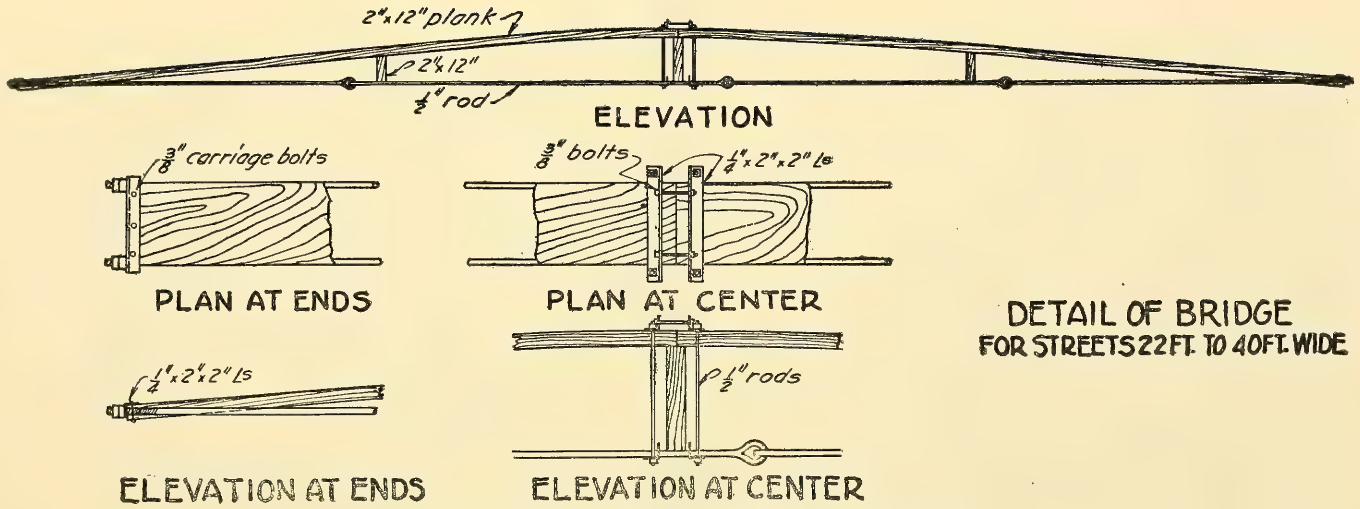


FIG. 23.—BRIDGE FOR PAVEMENTS MORE THAN 20 FEET WIDE.

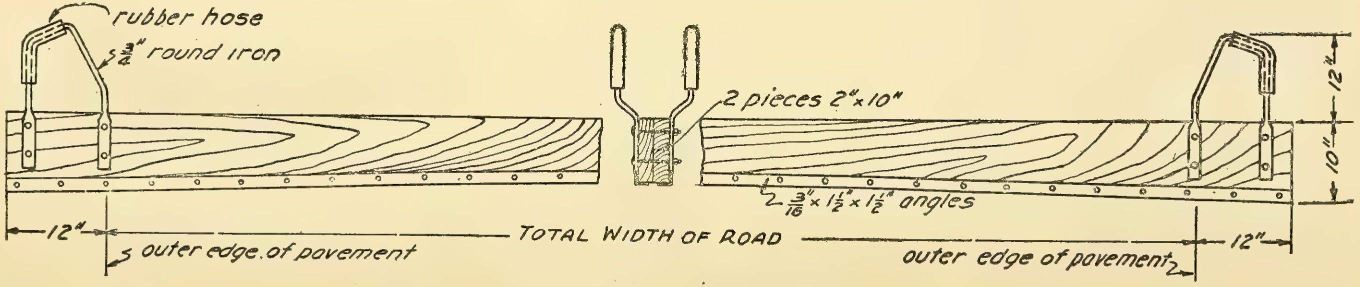


FIG. 24.—DETAIL OF STRIKEBOARD.

crete from freezing. If the concrete is partially hardened, protection may be provided by covering it with straw.

A 1-in. layer of sawdust, with a canvas overtop, has been found very satisfactory where the temperature does not drop more than 3° or 4° below freezing. Hay or straw used as a covering, covered over with canvas, will also serve the same purpose. It is important, where this is used, that the material be lightly forked onto the road so as to retain as much air as possible, which is the heat retaining medium.

Of the chemical compounds which have been used in cold weather construction work, calcium chloride probably gives the best results. Calcium chloride possesses the property of lowering the freezing point of water and accelerating the setting of cement. Laboratory tests show that small quantities of calcium chloride do not cause an appreciable reduction in the strength of the concrete, nor corrosion of the reinforcing steel.

This compound is generally sold in crystal form and is readily soluble in water. If used in quantities of about 3 per cent by weight of the mixing water it will enable the concrete to be placed with the temperature slightly below freezing with little danger of injury.

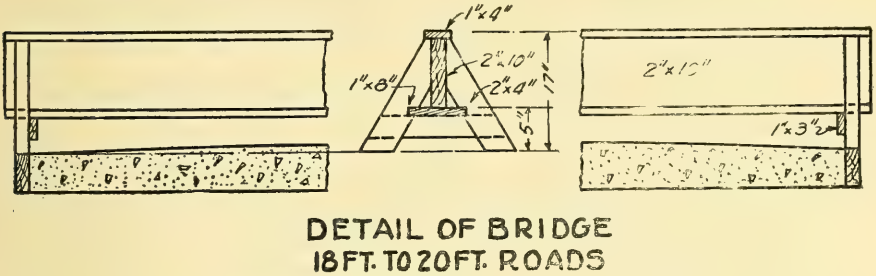


FIG. 25.—BRIDGE FOR PAVEMENTS UP TO 20 FEET WIDE.

It can best be added to the mix by making a solution having a concentration of 1½ lb. of calcium chloride per gallon of water. This solution may be kept in a barrel near the mixer and a gallon of it added for each sack of cement used. For the usual 1: 2: 3 mix this corresponds closely to 3 per cent of the total mixing water.

The use of this material will add about 3c. per sq. yd. to the cost of the concrete.

Attention is called to the fact that these special measures of heating aggregates, etc., should be adopted only after all ordinary precautions are in effect. The heating of the mixing water should not be counted upon to counteract the ill effects of dirty or poorly graded aggregate or an over supply of water in the batch.

The use of chemical compounds should not be solely depended upon to prevent freezing as they should be used only in conjunction with the other methods for insuring good concrete.

The strength of concrete is greatly affected by the temperature during the time of setting, as shown by an investigation carried out by Prof. A. B. McDaniels at the University of Illinois.

The comparatively low strength of concrete exposed to cold weather increases the importance of taking precautions to exclude traffic for a longer period of time from concrete roads finished at lower temperatures.

CHAPTER VII.

JOINTS AND REINFORCING.

32. TYPES.

Joints are of two general types, the expansion joint, where space is left which is filled with a plastic material, and the contraction joint, which divides portions of the concrete without any appreciable space being left between.

33. EXPANSION JOINTS.

Where expansion joints are used it is recommended that the space be not less than ¼ in. and not more than ½ in. and that some prepared form of filler be used which can be handled readily and which is of sufficient stiffness to permit placing.

All expansion joints should extend entirely through the pavement and through the integral curb when that is constructed with the pavement.

34. CONTRACTION JOINTS.

If contraction joints are to be made at points other than where the work has stopped temporarily as at noontime or the end of a day, what is known as the concealed joint is recommended. This is constructed by placing a thin sheet of material, preferably metal, at the place where the joint is desired, the plate extending to within 1 in. or less of the surface, thus permitting the road to be finished with the same facility as though no joint had been provided.

Instead of a thin sheet of metal a thickness of tar paper has been successfully used, but unless the joint is to extend entirely through the pavement and be of the expansion type, no appreciable thickness of compressible material should be used in this construction.

35. JOINT SPACING.

Joints should be placed at regular intervals. The theory of the expansion joint assumes that after the road is constructed the concrete slabs expand, due to increased temperature and moisture content. The expansion space is provided by the joint material. The tendency is to increase the spacing between joints and from 25 to 35 ft. has been the usual practice. This is being increased to 50 ft., which spacing is recommended. But where concealed contraction joints are used it is recommended that they be spaced 30 ft.

36. INSTALLATION OF JOINTS.

For installing expansion joints a wooden bulkhead cut to the exact cross-section of the pavement should be used. For pavements up to 20 ft. in width this may be cut from a single piece of 2-in. plank, while if the bulkhead is to be more than 20 ft. in length it should be in two sections, each of these sections being made up of two thicknesses of 1-in. dressed lumber.

A cord is stretched between forms at the desired location of the joint at right angles to the center line of the pavement. Two by four stakes are then driven and the bulkhead placed against them. Care should be taken to see that the height of the bulkhead at the curbs or sides of the pavement conforms to the desired elevation. If the bulkhead is made in two parts, as on wide streets, it will be necessary also to set the middle of it so as to insure the proper elevation at this point.

The joint material is then placed against the bulkhead and pinned down with steel pins on which there is an outstanding lug. A few shovelfuls of concrete are thrown against it to keep it upright. Concrete is then deposited up to this joint and luted or struck to place. The mixer is then moved ahead and concreting continued for about 8 to 10 ft.

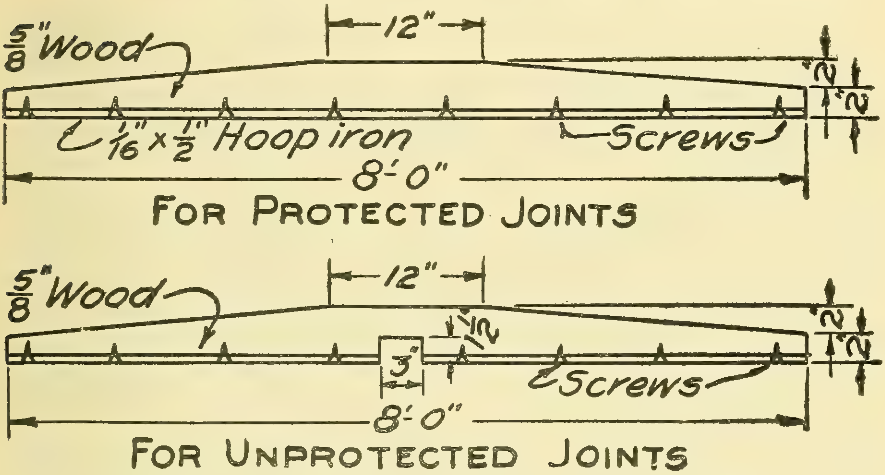


FIG. 26.—STRAIGHT EDGE FOR SECURING SMOOTH RIDING JOINTS.

(Steel protection plates for joints are not recommended, but if used should be tested for height with straight edge shown.)

The wooden stakes supporting the bulkhead are then removed and concrete worked under it as the bulkhead is lifted from one end. After the bulkhead is removed the surface adjacent to the joint is brought to grade as established on the other side of the joint with a small amount of additional material.

It will be noted that as the bulkhead is cut to the exact cross-section of the pavement it furnishes a precise mark or level for luting.

To insure that the concrete on both sides of the joint is at the elevation corresponding to the adjacent surface of the pavement a wooden straightedge, which can be made of ⅝-in. stock, 5 in. wide and about 8 ft. long, shod on the under side with 1/16-in. strap iron, should be worked at right angles to the joint, using a sawing motion of 2 or 3 in. A notch is necessary at the center of this straight-edge to provide for the joint material which projects above the surface of the pavement. (Fig. 26.)

By beginning at the center of the slab and working toward each edge the surplus concrete left by the strikeboard is swept off, producing a uniform surface on both sides of the joint.

This work should be done from the finisher's bridge and should follow up the strikeboard immediately after it has passed the joint.

A split-float should also be used to secure the final finish at the unprotected joints. (Fig. 27.)

Before the pavement is opened for traffic, the joint material which extends above the pavement should be trimmed so that not more than ⅜ in. of it is above the surface. By riveting cleats on the bottom of a shovel a tool is formed which accomplishes this with the minimum amount of labor.

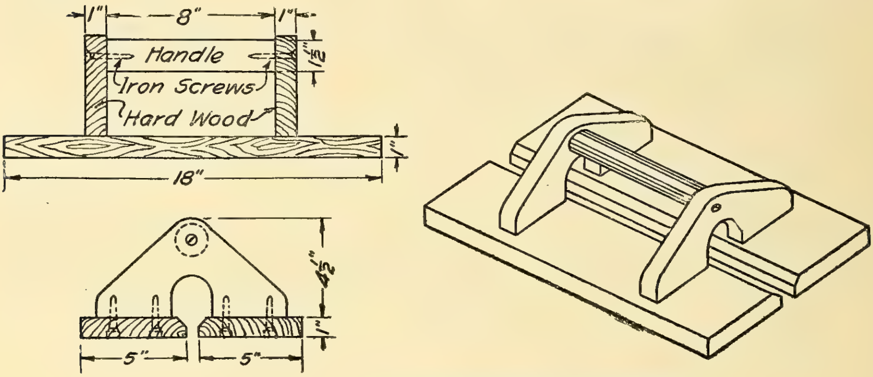


FIG. 27.—SPLIT FLOAT FOR FINISHING AT JOINTS.

The shovel is pushed against the joint material, trimming it to the height required. (Fig. 29.)

Around catch basins, manholes, inlets, telegraph poles and other structures that may protrude through the surface of a pavement it is recommended that ¼- to ½-in. expansion joint should be made. The filler for the joint should be of such material that it will remain in place when put around the catch basin or other structure protruding through the pavement, and should extend about ½ in. above the surface. The filler material is first put into position and then the concrete placed about it.

37. PLACING REINFORCEMENT.

For pavement reinforcement wire mesh or expanded metal weighing approximately 25 lb. per 100 sq. ft. is recommended.

The reinforcing should be placed not less than 2 in. from the finished surface of the pavement and should extend to within 2 in. of all joints, but not across them. Adjacent widths of the fabric should be lapped not less than 4 in. when the lap is made perpendicular to the center line of the pavement and not less than 1 ft. when the lap is parallel to the center line.

The effective areas of the reinforcing members at right angles to each other may vary between 1: 1 and 6: 1. Reinforcement should be placed so that the effective sectional area measured parallel to the center line of the pavement is at least as great as that measured at right angles to the center line. It is recommended that the former area be from three to five times the latter since the reinforcement will then offer greater resistance to the formation of longitudinal cracks.

It is essential that reinforcing be free from excessive rust, paint, or scale so that it will obtain a firm bond with the concrete.

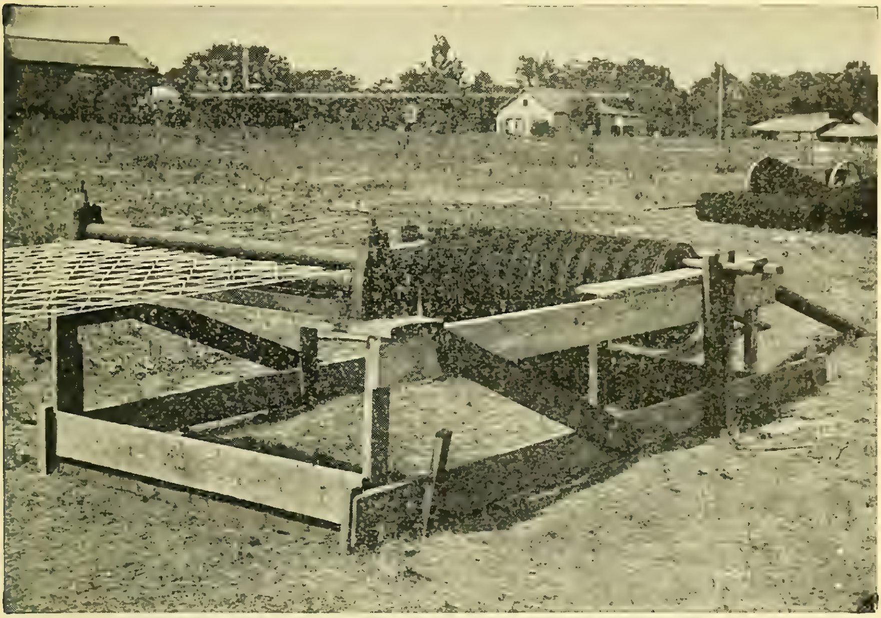


FIG. 28.—DEVICE FOR STRAIGHTENING REINFORCEMENT.

As wire mesh is delivered in rolls, it must be straightened before placing on the base course, to insure complete covering by two inches of wearing-course concrete. As the weight of reinforcement increases, this is a more important matter, and special apparatus must be provided to do the work. For this purpose the American Steel & Wire Co. has devised a straightening machine. The machine is set up on a wooden frame of sufficient height so that a roller will support the roll of reinforcement clear of the ground. A team hitched to the forward end of the roll pulls the mesh through the straightening machine. The mesh drops to the ground and is cut to required lengths. The contractor, on whose work the photo shown

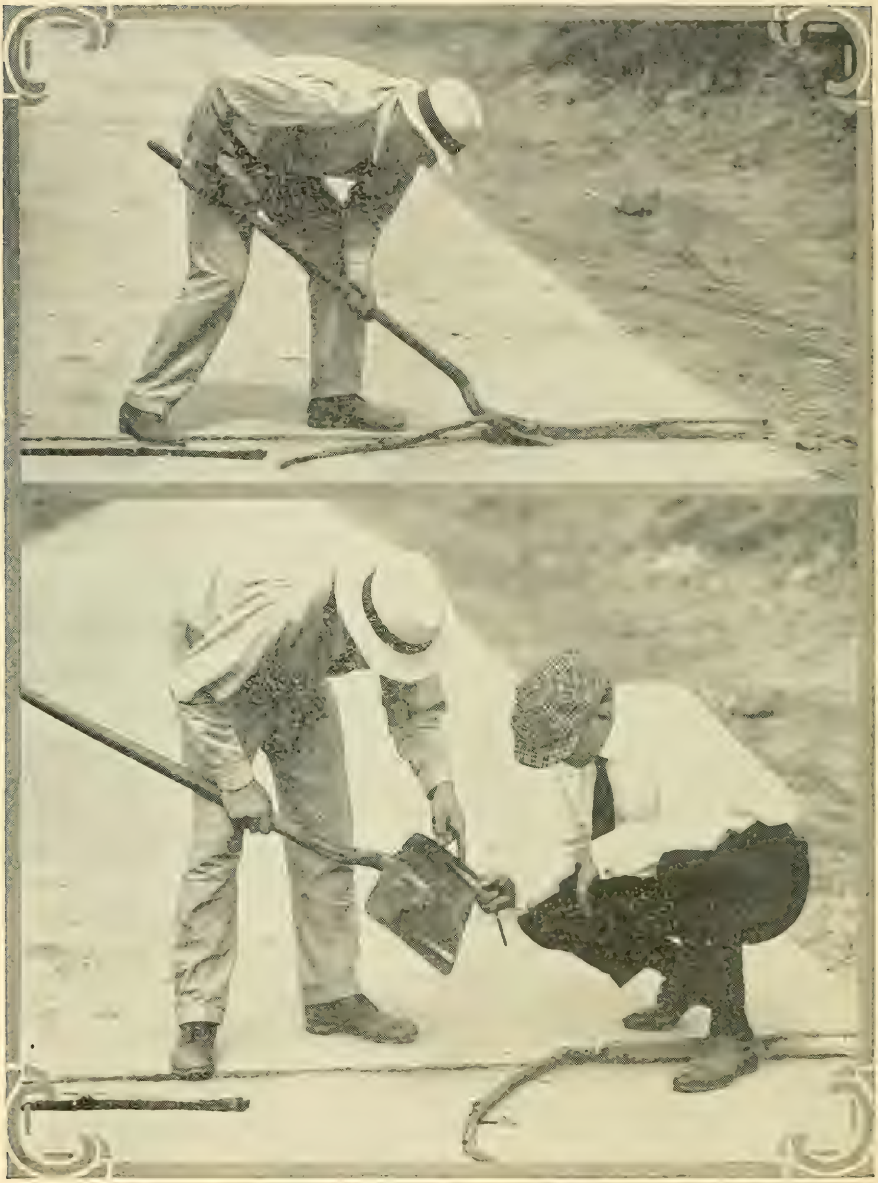
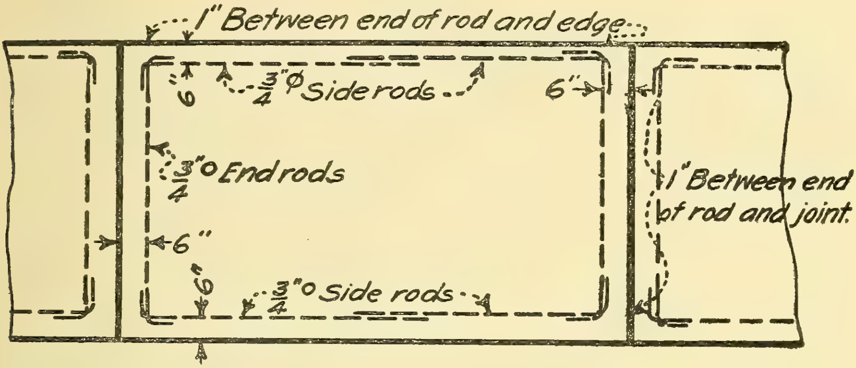


FIG. 29.—TRIMMING JOINT FILLER.

was taken, also hauls six to eight lengths of reinforcement by team to the point on the work where they are to be used. The usual method is to cut a number of lengths of the reinforcing material, place them in one pile on the roadbed and iron them straight with a steam roller.

38. CIRCUMFERENTIAL REINFORCING.

This method of reinforcing concrete roads is comparatively new, but has been used successfully in a few pieces of work. It consists in laying around the edge of the slabs a single steel bar ¾ in. round or ⅝ in. square. The object of concentrating the reinforcement at the edges is to place it where it has been observed that cracks usually originate. Fig. 30 and Table 2 show the method of placing the rods and the amount of steel required for each slab on 18-, 20- and 24-ft. pavements with varying joint spacing.



1" Between end of rod and edge
6"
$\frac{3}{4}$"φ Side rods
6"
$\frac{3}{4}$"o End rods
6"
6"
$\frac{3}{4}$"o Side rods
1" Between end of rod and joint

NOTE· Reinforcement to be placed halfway between top and bottom of slab.

FIG. 30.—METHOD OF PLACING CIRCUMFERENTIAL REINFORCING.

CHAPTER VIII.

FINISHING.

39. MACHINE FINISH.

Two methods of finishing are used, machine finish and hand finish. Machine finish is to be recommended where the machine is such that it requires a stiff mixed concrete to work successfully. A machine requiring concrete of a thin consistency for operation is not recommended.

40. ROLLER AND BELT METHOD.

The roller and belt method of finishing is recommended where machines are not used. The following manipulation is recommended:

As soon as possible after the concrete has been struck off, it shall be rolled with an approved metal roller having a smooth, even surface, approximately 6 ft. in length, not less than 8 in., nor more than 12 in. in diameter, and weighing not more than 100 lb. (Fig. 31-32.) On pavements less than

20 ft. in width, the roller may be operated with a handle, which shall be at least 2 ft. longer than the width of the pavement, and all rolling shall be done from one side of the slab. On pavements 20 ft. and more in width, the roller shall be provided with two bails, to which ropes shall be attached and the roller pulled across the pavement. The roller shall be operated at such an angle with the center line of the pavement that it advances along the pavement about 2 ft. for each time across. The roller shall pass from one edge of the pavement to the other, care being taken not to run the roller over the side forms so that earth or other foreign material will adhere to it. After the roller has covered a given area in the manner described, the same area shall be similarly covered by the roller not less than three times at intervals of 15 to 40 minutes, and as many times additional as may be necessary to remove excess water.

QUANTITIES AND LENGTHS OF $\frac{3}{4}$" ROUND RODS*
FOR 1 CONCRETE SLAB

DISTANCE BETWEEN JOINTS IN FEET.	WIDTH OF ROAD IN FEET.								
	18'			20'			24'		
	NUMBER AND LENGTH OF END PIECES	NUMBER AND LENGTH OF SIDE PIECES	TOTAL WEIGHT OF BOTH PER SLAB POUNDS	NUMBER AND LENGTH OF END PIECES	NUMBER AND LENGTH OF SIDE PIECES	TOTAL WEIGHT OF BOTH PER SLAB	NUMBER AND LENGTH OF END PIECES	NUMBER AND LENGTH OF SIDE PIECES	TOTAL WEIGHT OF BOTH PER SLAB POUNDS
25	2 18'-0"	2 25'-0"	129	2 20'-0"	2 25'-0"	135	2 24'-0"	2 25'-0"	147
35	2 18'-0"	2 35'-0"	159	2 20'-0"	2 35'-0"	165	2 24'-0"	2 35'-0"	177
40	2 18'-0"	4 21'-0"	180	2 20'-0"	4 21'-0"	186	2 24'-0"	4 21'-0"	198
50	2 18'-0"	4 26'-0"	210	2 20'-0"	4 26'-0"	216	2 24'-0"	4 26'-0"	228
75	2 18'-0"	6 26'-6"	293	2 20'-0"	6 26'-6"	299	2 24'-0"	6 26'-6"	311
100	2 18'-0"	8 26'-6"	372	2 20'-0"	8 26'-6"	378	2 24'-0"	8 26'-6"	390

* $\frac{5}{8}$" □ Bars may be substituted, reducing weights to 88½% of those shown

TABLE 2.—MATERIAL REQUIRED FOR CIRCUMFERENTIAL REINFORCING.

After the rolling has been completed the pavement shall be finished by two applications of a belt made of canvas or rubber belting, not less than 6 in. wide and not less than 2 ft. longer than the width of the pavement. The belt shall be applied with a combined crosswise and longitudinal motion. For the first application vigorous strokes at least 12 in. long shall be used, and the longitudinal movement of the belt along the pavement shall be very slight. The second application of the belt shall be immediately after the water glaze or sheen disappears, and the stroke of the belt shall be not more than 4 in. and the longitudinal movement shall be much greater than for the first belting.

The finished surface of the concrete shall not vary from a line parallel to the pavement center line more than ¼ in. in any 3-ft. length when tested with a gage with which all variations can be measured accurately, and any allowable change in elevation shall be gradual.

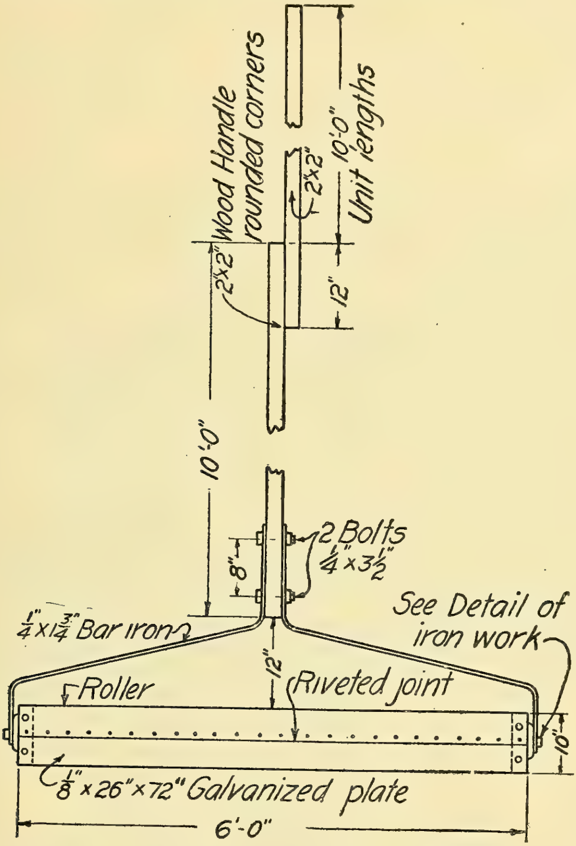


FIG. 31.—DETAIL OF ROLLER FOR FINISHING.

At every point in any cross-section of the pavement the finished surface of the concrete shall be convex upward.

When the belting has been completed the portion of the surface next to the forms should be smoothed if necessary with a wooden float and the edge of the pavement rounded to a radius of 1 in. with a suitable edging tool.

CHAPTER IX.

CURING.

41. EARTH COVERING.

The method which has been used more than any other in curing concrete pavements is to keep them covered for at least ten days with a layer of not less than 2 in. of earth, which is kept moist by sprinkling. As soon as the concrete has been finished with the final belt, it is covered with canvas

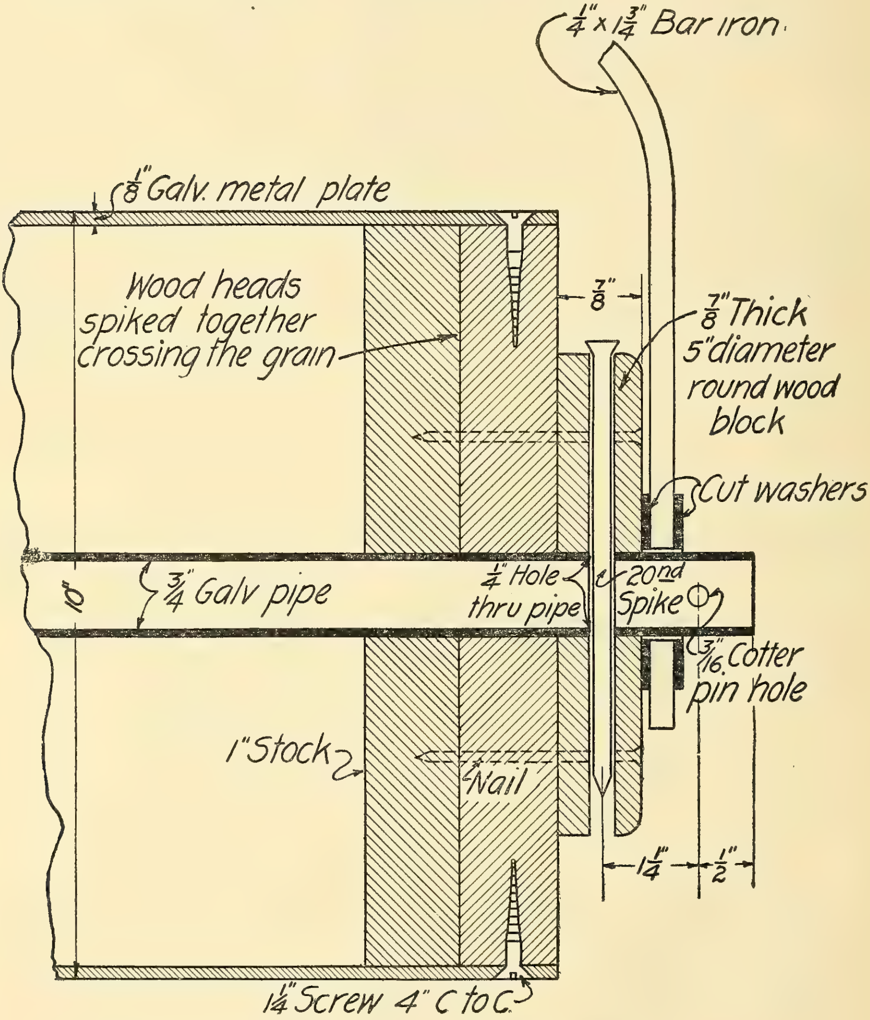


FIG. 32.—DETAIL OF ROLLER FOR FINISHING.

to prevent rapid evaporation. As soon as it has hardened sufficiently to permit earth to be thrown on without damaging the surface, this is done and the earth sprinkled at regular intervals to insure a moist covering. After a period of 10 days this covering is removed, but the pavement is kept closed to traffic for at least four days more and in some cases 10 days to two weeks longer if the temperature is below 50° F. to insure thorough curing.

When the average temperature is below 50° F., it will usually be found desirable to omit the earth covering and sprinkle the pavement only when the concrete shows signs of drying out too rapidly. Except on days when the sun is unusually warm, sprinkling night and morning will be sufficient. Hardening shall be allowed to progress slowly and uniformly, but shall not be so much retarded that opening of the pavement to traffic in a reasonable length of time is prevented.

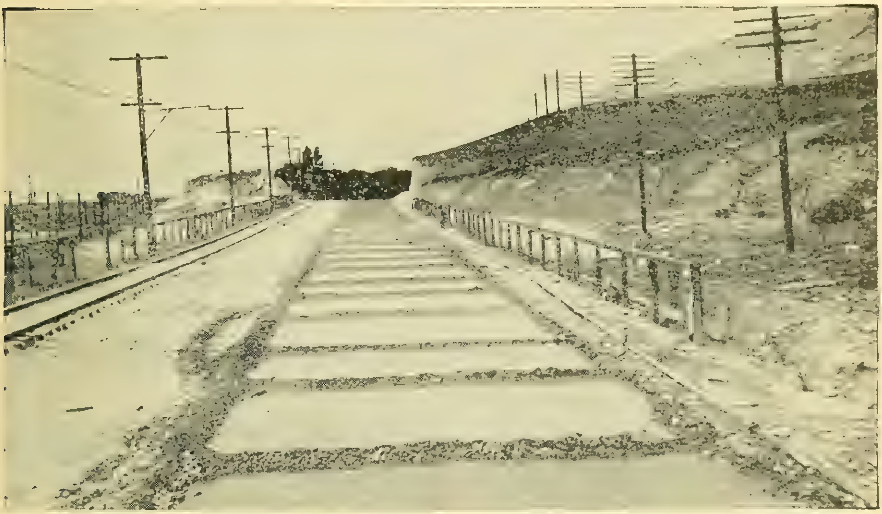


FIG. 33. PAVEMENT COVERED WITH WATER RETAINED BY DAMS FOR CURING.

42. Ponding.

Another method of curing concrete pavements, which has proved eminently satisfactory is by keeping them covered with about 2 in. of water, which is retained by earth dams built across the pavement at frequent intervals. (Fig. 33.)

After finishing, the concrete is covered with canvas until it has hardened sufficiently to permit building the dams across the pavement, which are generally placed at the joints and along the edges. If the pavement is one on which curbs are being constructed, longitudinal dikes are unnecessary. Where there is no curb, some contractors have preferred to build two longitudinal dams at each edge of the pavement and especially on wide streets, as the breakage of the dikes with the consequent loss of water is less likely.

Flooding is generally done in the evening when the water is not needed for the mixer, and it is kept at a minimum depth of 2 in. at the crown for at least 10 days.

After the pavement has cured sufficiently with the water covering, the dikes are broken and the water allowed to run off, leaving the surface clean.

In order to protect the pavement from traffic, it is necessary that suitable barricades be erected and, where it is necessary, detour signs shall be put up so as to cause traffic the least inconvenience possible.

CHAPTER X.

MAINTENANCE.

Maintenance should be systematic and imperfections given immediate attention.

43. CRACKS AND JOINTS.

The crack or joint should first be cleaned with a sharp hooked pick and stiff brush, care being taken to remove all loose particles. If the opening is too narrow to permit cleaning in this manner, it should be cleaned with an air jet from an automobile tire pump. After thorough cleaning, tar should be poured into the crack in sufficient quantity, just to flush over the edges. The tar should then be covered with coarse dry sand. It will usually be found desirable to remove old joint material to a depth of ¼ to ½ in. below the surface of the concrete, so that new repair materials will have a good bond.

Care shall be taken not to pour the tar in such quantities that an unsightly wide strip is made on the surface. By bending the spout of the tar pot so that a long narrow opening is provided, the tar can be poured in a very thin, narrow stream. It needs but a little attention to pour cracks so that a narrow ribbon of tar 1 to 2 in. in width is left.

44. TAR.

Refined coal tar should be used, having a melting point (½-in. cube method in water) of about 100° F. The tar should be heated from 225° to 250° F., at the time of application and may be applied by means of a sprinkling can with spray nozzle removed.

45. SURFACE POCKETS.

A pocket in the surface, 1 to 3 in. in size, due to soft aggregates or the disintegration of a lump of clay, or a piece of coal or wood, should first be thoroughly cleaned. It should then be filled with hard, small aggregates, ranging in size from ¼ to ⅜ in. Hot tar should then be poured into the opening until the tar is flush with the surface. Very coarse, dry sand should then be sprinkled on the tar immediately.

A cold mix of small stone and emulsified asphalt has been successfully used for this type of repair work.

46. SLIGHT DEPRESSIONS.

If for any cause a slight depression has formed, it can be coated with tar, very coarse sand added, covered with tar, and the whole covered with sand and tamped into place.

47. REPLACEMENTS.

If it is necessary to cut a hole through the entire thickness of the concrete slab, gravel should be placed in the subbase and thoroughly rammed, so as to form a compacted base on which the new concrete will rest. Where water has been allowed to stand in such a place, it should be compacted after the water has been removed and just before laying the concrete. Before placing the concrete the sides of the opening should be painted with a mixture of neat cement and water.

The consistency of the concrete should be sufficiently stiff to require considerable tamping to bring water to the surface so that it may be possible to ram it thoroughly into place. A small tamp, made of a 2 × 1-in. stick, 18 in. long, should be used to ram the concrete adjacent to the edges.

After the concrete is rammed into place it should be struck true to the surrounding surface. A repair made in this manner cannot be told from the old pavement after a few months' use.

A new patch should be kept moist for at least four or five days, and protected from traffic at least 10 days.

Equipment that has been found to work very satisfactorily and economically for the maintenance of concrete roads by the Illinois State Highway Department is shown in Figs. 34-35.

CHAPTER XI.

RESURFACING OLD CONCRETE PAVEMENTS.

Where an old concrete pavement is to be resurfaced, care must be taken to clean the pavement thoroughly. Grout of wet consistency should be thoroughly brushed over the old surface and every effort made to secure a bond between the two layers of concrete. In placing, finishing, and curing the new concrete the recommended practice for concrete pavements should be followed. Joints should be provided in the top layer directly over those in the pavement below. These can be made by placing wooden strips over the old joints when new concrete is placed. After the resurfacing has hardened sufficiently to allow the removal of these strips the space left is filled with tar or other joint filler.

The thickness of the resurfacing layer should not be less than 3 in. and should be mixed in the proportions of 1: 1½: 2½, using particles ¼ to 1 in. for the coarse aggregate. Steel reinforcement weighing at least 25 lb. per 100 sq. ft. should be placed in the middle of the resurfacing layer.

If there are any holes in the surface of the old pavement these should be thoroughly brushed out and filled with 1: 2: 4 concrete before placing the top surface.

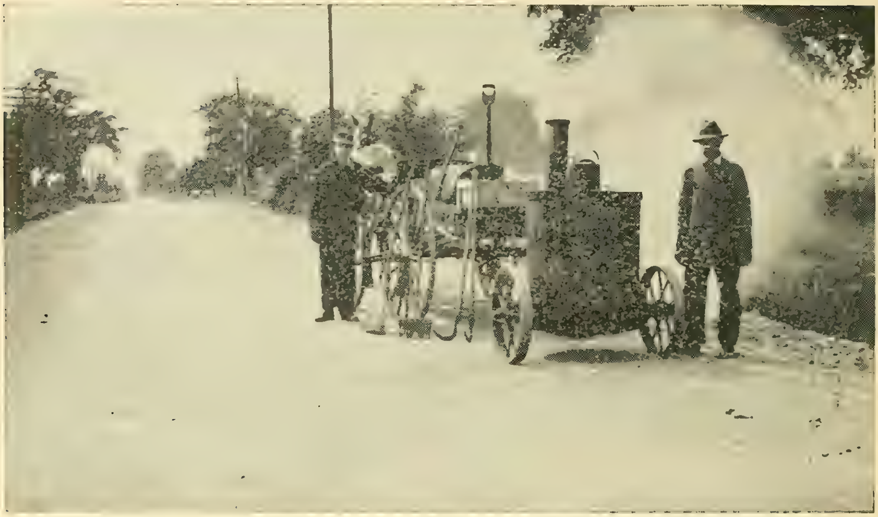


FIG. 34.—MAINTENANCE OUTFIT OF ILLINOIS DIVISION OF HIGHWAYS.

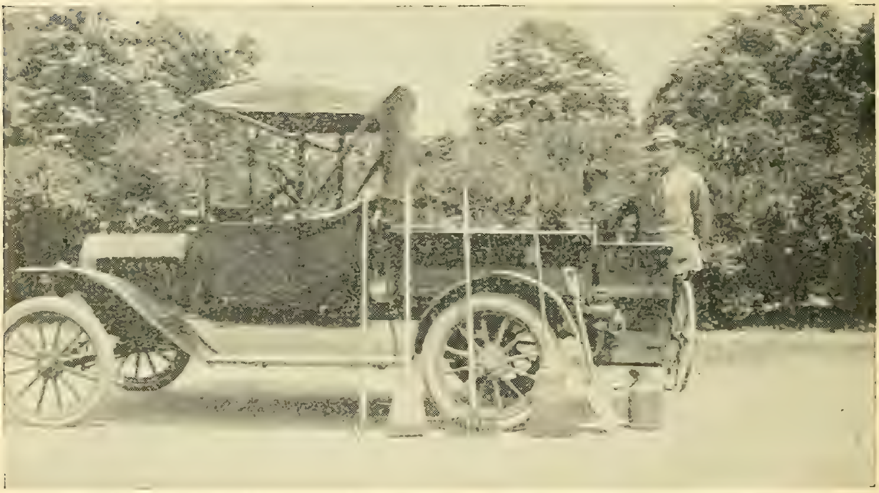


FIG. 35.—FORD TRUCKS ARE ALSO USED BY THE ILLINOIS DIVISION OF HIGHWAYS FOR MAINTENANCE. A KETTLE FOR HEATING TAR IS ATTACHED TO THE REAR OF THE TRUCK.

Where it is desired to widen the pavement in conjunction with the resurfacing, concrete of 1: 2: 4 mix should be placed along the edges of the slab to the width desired and of the thickness of the old slab. The edges of the slab should be thoroughly wetted to insure a bond.

This concrete should then be struck off to the surface of the old slab and allowed to cure for two days, during which period it should be kept wet. The resurfacing is then placed on the widened slab as previously outlined.

REPORT OF COMMITTEE ON CONCRETE HIGHWAY BRIDGES AND CULVERTS.

Owing to the unusual burdens thrown upon the members of this committee due to the enlistment of associates, employees and assistants in the United States Army, your committee begs leave to submit that it has not been able to formulate a definite report or recommendations.

The following matters are under advisement and it is hoped that during the ensuing year these subjects, as well as others, may be treated by the committee in an appropriate manner:

1. A review of highway load assumptions to be used in the design of concrete highway bridges and culverts.
2. The distribution of concentrated loads on simple slab superstructures having no earth covering.
3. The distribution of concentrated loads on floor slab and girders of through girder bridges having no earth covering.
4. The distribution of concentrated loads on the floor slab and beams of T-beam and deck-girder bridges having no earth covering.
5. The distribution of concentrated loads through earth overburden of various depths.
6. Design assumptions concerning the treatment of earth pressures on abutment and wing walls.

Any suggestion concerning these or other matters pertaining to concrete highway bridges and culverts made by the Institute or any of its members will gladly be received by the committee.

CLIFFORD OLDER, *Chairman.*

PROPOSED STANDARD SPECIFICATIONS FOR FIRE TESTS OF MATERIALS AND CONSTRUCTION.*

I. Control of Fire Tests.

Time-Temperature Curve.

1. The conduct of fire tests of materials and construction shall be controlled by the standard time-temperature control curve shown in the accompanying drawing. The points on the curve which determine its character are,

1000° F. at 5 minutes
1300 " " 10 "
1550 " " 30 "
1700 " " 1 hour
1850 " " 2 "
2000 " " 4 "
2300 " " 8 "

Determination of Temperatures.

2. (*a*) The temperature fixed by the curve shall be deemed to be the average true temperature of the furnace gases as obtained from the readings of several thermo-couples (not less than three) symmetrically disposed and distributed in such a manner as to show the temperatures of the gases near all parts of the sample.

(*b*) The temperatures shall be read at intervals not exceeding 5 minutes during the first hour, and thereafter the intervals may be increased to not more than 15 minutes.

II. Classification as Determined by Test.

Classification.

3. (*a*) Fire-resistive materials and construction shall be classified in accordance with the degree of protection they afford when measured by a

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* This specification was prepared by a conference of representatives of the following technical organizations:

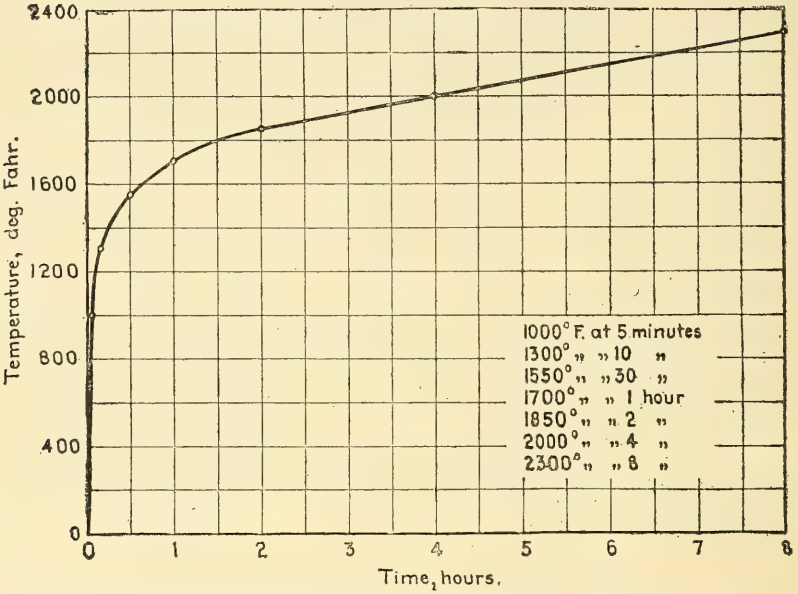
American Society for Testing Materials.
National Fire Protection Association.
U. S. Bureau of Standards.
National Board of Fire Underwriters.
Underwriters' Laboratories.
Associated Factory Mutual Fire Insurance Companies.
American Institute of Architects.
American Society of Civil Engineers.
American Society of Mechanical Engineers.
Canadian Society of Civil Engineers.
American Concrete Institute.

At the Annual Convention, 1918, it was ordered printed with the statement that it is the purpose of the Institute to make it a standard specification when it can take the usual course prescribed in the by-laws for the adoption of standards.

fire test conducted in conformity with the standard time-temperature control curve as,

4-hour Protection
2 " "
1 " "
½ " "
¼ " "

(*b*) Other classes may be interpolated or added as needed.



STANDARD TIME-TEMPERATURE CONTROL CURVE FOR FIRE TESTS OF MATERIAL AND CONSTRUCTION.

III. TEST STRUCTURES.

Test Structures.

4. (*a*) The test structure may be located at any place where all the necessary facilities for properly conducting the test are provided.

(*b*) Entire freedom is left to each investigator in the design of his test structure and the nature and use of fuel, provided the test requirements are met.

IV. TEST SAMPLES.

Test Samples.

5. The material or construction constituting the test sample shall be truly representative of the regular practice.

Loading.

6. For any material or construction intended to carry load other than its own weight, at least the full rated safe working load shall be applied during the entire fire test.

V. CONDUCT OF FIRE TESTS.

Duration of Test. 7. The fire test on the sample with its applied load, if any, shall be either (*a*) continued till failure occurs, or (*b*) stopped at the period for which classification is desired, allowed to cool and loaded to failure.

Application of Water. 8. (*a*) A second and separate test shall be made to determine the effect of a hose stream on a sample under fire test, the water being applied not later than one hour after the beginning of the test.

(*b*) The size of nozzle, water pressure and time of water application shall be as indicated in Table I. The hose stream shall be first directed at the middle of the sample and then at all parts of the exposed faces, changes in direction being made slowly.

TABLE 1.

Parts of Structure.	Type of Protection.	Size of Hose Nozzle, in.	Water Pressure at Nozzle, lb.	Time of Application, min.
Floors and Roofs	4 hour	1⅛	50	10
	2 "	1⅛	50	5
	1 "	1⅛	50	2.5
Walls, Columns and Partitions	4 "	1⅛	50	5
	2 "	1⅛	30	3
	1 "	1⅛	30	2.5

VI. FLOOR AND ROOF TESTS.

Size of Sample. 9. For floor and roof tests the sample shall be of such a size that the minimum span of the supporting beams of the floor arch shall be 12 ft., and the supporting beams and girders shall have a clearance of at least 8 in. from the walls of the test structure.

Time of Testing. 10. The floor may be tested as soon after construction as desired, but within 40 days. Artificial drying will be allowed if desired.

Plastering. 11. If the construction is to be plastered in practice, the sample shall be plastered in the same manner.

Loading. 12. The floor shall be loaded so as to develop in each member of the construction stresses up to the maximum working stress allowed in the material of the member.

Requirements. 13. The test shall not be regarded as successful unless the following conditions are met:

(*a*) No fire shall have passed through the floor or roof during the test.

(*b*) The floor or roof shall have safely sustained the full rated safe working load during the test.

(*c*) After a fire test that has not been carried to failure, or after a combined fire and water test, the floor or roof shall sustain safely the dead load plus at least 2½ times the designed live load applied not less than 24 hours nor more than 72 hours after the completion of the fire test.

(*d*) If the fire test has been continued to failure, the failure shall not have occurred within a period of 25 per cent in excess of the period for which classification is desired.

VII. NON-BEARING PARTITION TESTS.

Size of Sample. 14. For partition tests the area of the sample shall be not less than 100 sq. ft. and no dimension less than 9 ft.

Determination of Temperatures. 15. Temperatures on the outer surface of the sample shall be read by thermometers, not less than five, symmetrically disposed and placed against the surface of the sample with their bulbs properly protected against radiation of heat.

Position of Nozzle. 16. The distance of the nozzle from the sample during the application of water shall be not more than 20 ft. when the hose stream is applied approximately normal to the surface of the sample, which distance shall be reduced by 1 ft. for each 10 deg. of angle from the normal when the hose stream is applied at an angle to the surface of the sample.

Requirements. 17. The test shall not be regarded as successful unless the following conditions are met:

(*a*) No fire shall have passed through the partition during the fire test.

(*b*) Transmission of heat through the sample shall not have been such as to raise the temperature on the outer surface of the sample in excess of 300° F.

(*c*) The partition shall have safely sustained the pressure of the hose stream.

(*d*) The partition shall not have warped or bulged or disintegrated under the action of the fire or water to such an extent as to be unsafe.

AMERICAN CONCRETE INSTITUTE.

STANDARDS NO. 5, 17, 18, 19 and 20.

ADOPTED BY LETTER BALLOT, AUGUST 30, 1918.

CHANGES IN STANDARD SPECIFICATIONS FOR CONCRETE ROADS, STREETS AND ALLEYS.

(The page numbers refer to the Proceedings of the American Concrete Institute for 1917, Volume XIII.)

Pages 425, 433, 441, 450, 458.

Change second sentence in "Coarse Aggregate" to read: "The size of the coarse aggregate shall be such as to pass a three (3) inch round opening and shall range from not less than one and one-half (1½) inches down. Not more than five (5) per cent shall pass a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed."

Pages 425, 433, 441, 450.

Under "Reinforcement" insert the words "steel and" between "be" and "free."

Pages 428, 436, 444, 453.

Under "Joints," "Width and Location" change first sentence to read "Transverse joints shall be placed across the pavement and shall be in width and distance apart as shown on the plans."

Page 426.

In drawing reduce radius of curve at edge of pavement to show use of edger only.

Pages 434, 442.

In drawing, do not show reinforcement extending into integral curb.

Pages 430, 438, 446, 454.

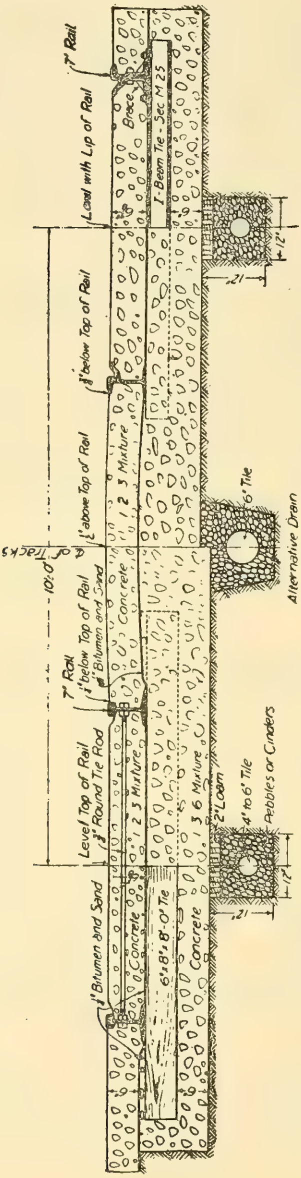
Under "Reinforcing" transfer last phrase in section beginning "and in most cases," etc., to complete "Note" in same section.

Page 459.

Under "In Track," change third paragraph to read, "When T-rails are used, a flangeway extending 1⅛ in. below the top of the rail shall be formed along each rail. Between flangeways the surface of the concrete shall be level with the top of T-rails."

Page 460.

Change drawing as shown herewith.



CONCRETE PAVEMENT FOR RAILWAY TRACKS.

Adapted to Type D Track Construction. As recommended by Committee on Way Matters (1914). Am. Electric Ry. Eng. Assn.

ANNUAL REPORT OF BOARD OF DIRECTION.

American Concrete Institute.

The headquarters of the Institute for the past year have been in Boston with Henry B. Alvord in charge, first as Administrative Assistant beginning in February, 1917, and later as Secretary when Lieut. Harold D. Hynds resigned, in October, 1917, to enter the military service of the United States.

The Institute has published Vol. IX, Vol. X and Vol. XIII since the last convention and also a membership pamphlet.

The Treasurer's report, as shown by the Auditor's report, will show that previous to the last convention there was a cash balance of $3,922.16, whereas on May 31, 1918, the blance was $4,378.24.

Our membership has increased during the year despite the war conditions prevailing. We desire, however, to urge upon the membership the necessity of continued support, if the Institute is to maintain its power of usefulness to the concrete industry.

Respectfully submitted,
The Board of Direction.
W. K. Hatt,
President.

ANNUAL REPORT OF TREASURER.

At the annual convention the Treasurer submitted the report of the auditors, Cooley & Marvin Co., Accountants and Engineers, of Boston, Mass., for the twelve months ending May 31, 1918. This report is as follows:

Boston, Mass., June 24, 1918.

The American Concrete Institute,
6 Beacon St., Room 428, Boston, Mass.

Dear Sirs:

In accordance with your instructions we have made an examination of the books and records of the American Concrete Institute for the period from Feb. 1, 1917, to May 31, 1918, for the purpose of verifying the cash transactions of the period and presenting the financial condition of the American Concrete Institute at that date.

We submit herewith two exhibits, as follows:

Exhibit A. Statement of Condition as at May 31, 1918.
Exhibit B. Statement of Receipts and Disbursements for the period from Feb. 1, 1917, to May 31, 1918.

The cash in bank, amounting to $4,378.24, as shown by the cash book was verified by reconciliation with the statement rendered by the depositary as at May 31, 1918. During our examination paid checks, all of which were properly approved by your President and Secretary or Acting Secretary, were seen for all disbursements.

The remaining items on the statement of condition are shown in accordance with the records, and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year, and have restricted our examination to accounting for the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for, and that we have seen satisfactory evidence of payment for all disbursements.
2. That the cash in bank, amounting to $4,378.24 at May 31, 1918, was on hand at that date.
3. That the Statement of Condition (Exhibit A) is in accordance with the records and, subject to the foregoing comments, in our opinion properly presents the financial condition of the American Concrete Institute at May 31, 1918.

Very truly yours,

COOLEY & MARVIN CO.

EXHIBIT A.

STATEMENT OF CONDITION

As at May 31, 1918.

ASSETS.

Cash in bank		$4,378.24
Accounts receivable: dues		766.00
Inventories:		
Journals	$242.40	
Proceedings	827.50	
Supplies	45.35	
Total inventories		1,115.25
		$6,259.49

LIABILITIES AND SURPLUS.

Membership dues paid in advance	$15.00
Surplus	6,244.49
	$6,259.49

EXHIBIT B.

STATEMENT OF CASH RECEIPTS AND DISBURSEMENTS

As shown by the Cash Book
For the period from February 1, 1917, to May 31, 1918.

RECEIPTS.

Balance February 1, 1917		$3,922.16
Dues	$5,250.00	
Subscriptions	1,450.00	
Proceedings	1,028.14	
Bindings	384.36	
Interest on bank deposit	80.45	
Miscellaneous	67.56	
Far Rockaway investigation	300.00	
Total receipts		8,560.51
		$12,482.67

DISBURSEMENTS.

Proceedings	$4,880.79	
Membership campaign	273.90	
Office expense	1,589.21	
Board Meeting expense	193.84	
Office printing	71.25	
Convention	663.56	
Miscellaneous expense	231.88	
Far Rockaway investigation	200.00	
Total disbursements		8,104.43
Balance May 31, 1918—Exhibit A		$4,378.24

ABSTRACT OF MINUTES OF MEETINGS OF THE BOARD OF DIRECTION.

Meeting of Board of Direction, Held in Office of the Turner Construction Co., New York City, April 12, 1917.

Present: Hatt, Lesley, Turner, Anderson, Boyer, Gow, Wight, Alvord, and Hynds.

Minutes of Board meeting held February 10, stood approved.

Treasurer's report approved.

Frank C. Wight submitted report covering his activity as editor for the American Concrete Institute publications.

The policy of the Institute regarding the reprinting and publication in the Institute's proceedings of papers which were printed in other societies' reports, was discussed. The Board moved and seconded that action in this connection be left to the Executive Committee and the editor to decide.

Mr. Turner, Chairman of the Sub-Committee on Headquarters and Permanent Secretary, reported that it was not possible to have the secretarial duties of the Institute co-ordinated with the duties of any other technical society. Neither was it possible to procure permanent office location without considerable expense. On motion, Mr. Turner's report was accepted and the committee continued to further investigate the possibility of permanent office location and permanent secretary either in Chicago or New York.

President Hatt submitted a resolution of the Institute to the National Government, copy of which is attached, covering the Institute's co-operation as a technical society for the propagation of national defense and activities of war.

The project for raising funds for research work was discussed. Prof. Hatt emphasized the importance of this work and, on motion, the President was authorized to proceed with his plan to arrange to secure the money and select a committee to submit a program for the carrying out of this work, which must be submitted and approved by the Executive Committee.

Resolution adopted by the Board of Direction of the American Concrete Institute, at the meeting held on April 12, 1917:

That the Board of Direction of the American Concrete Institute, an organization whose activities are concerned with concrete construction, expresses the desire to render service to the Government of the United States in the present national crisis, in whatever fields of co-operation may seem appropriate, whether in counsel or in the supply of a list of names of its membership who are fitted for specific duties;

That the Secretary be instructed to extend this resolution with a list of the membership and copies of the Standards of the Institute to the Council of National Defense, the Chief of Engineers of the United States Army, the Secretary of the Navy.

MEETING OF BOARD OF DIRECTION, HELD AT TRAYMORE HOTEL, ATLANTIC CITY, N. J., JUNE 27, 1917.

Present: Hatt, Turner, Humphrey, Boyer, Lesley, Wight, Alvord and Hynds.

Minutes of the Board meeting of April 12th were approved.

Budget covering the finances of the Institute for next year was read by Mr. Alvord.

Mr. Wight, editor, reported that the Proceedings of the Thirteenth Annual Convention of the Institute were complete and in the printer's hands, excepting one discussion.

Mr. Humphrey reported that Vol. IX was complete and mailed to the membership; Vol. X completely edited and in the printer's hands.

Wm. M. Kinney was elected to the Board of Direction of the Institute, to succeed Ernest McCullough of the Fifth District, resigned.

The time and place of holding the next convention was discussed. On motion, subject to cancellation by the President or Secretary, in case of national expediency, it was decided that it should be held at Hotel La Salle, Chicago, at the usual time and should if possible be at the same time as other Convention meetings held in Chicago in February.

The new Committee on Concrete Barges and Ships was appointed with H. C. Turner as chairman. Committee consists of five members to be selected by the chairman. The committee is to cooperate with similar committees of other organizations for careful and just consideration of the use of concrete for ship and barge construction.

MEETING OF THE BOARD OF DIRECTION, AT THE OFFICES OF THE TURNER CONSTRUCTION CO., N. Y., ON OCT. 15, 1917.

Present: Hatt, Thompson, Humphrey, Turner, Lesley, Boyer, Kinney, Alvord, Hynds.

Minutes of the last meeting of the Board, held June 27th last, at Atlantic City, were dispensed with motion, and stood approved.

Treasurer's report showing a surplus of $2799.47 was accepted approved.

Mr. Wight, editor, in his absence requested the Secretary to state that all editorial work for which he has been responsible, is complete to date and the Board unanimously moved that a vote of thanks and appreciation be extended to Mr. Wight for his work in the past year for the Institute.

The Board also unanimously moved that thanks and appreciation be extended to Richard L. Humphrey for his work in connection with the editing of the back volumes of the Proceedings of the Institute.

President Hatt opened the meeting for discussion regarding the advisability of holding a convention this year and the Board unanimously voted to hold the convention and appointed three days of Thursday, Friday and Saturday, Feb. 7, 8 and 9. for the convention, and designated same to be held at Hotel La Salle, Chicago.

On motion, the Board instructed President to appoint a committee of three from the Board to assist the Secretary in forming a programme and procuring papers for the coming convention.

On motion, the President was instructed to appoint a committee to report at the next Board meeting, concerning a standard policy to be pursued by the Institute for numbering its standards.

Mr. Humphrey made a report of progress of the Far Rockaway Fire Committee, stating that the report was now being written on the field inspection and necessary data which had been completed—that the Bureau of Standards were assisting in tests on behavior of aggregates in fire, so that report of the committee can be conclusive.

The Board instructed that the report be preprinted and not given to the publications until after delivery at the Institute.

The Board instructed the Secretary to remit dues to those members in active service of the United States Army or Navy, should those members request same.

Mr. Hynds offered his resignation as Secretary, because of expected service in the United States Army, and same was accepted by the Board, to take effect at the pleasure of the President. The Board elected Mr. Alvord as Secretary, after acceptance of Mr. Hynds' resignation.

The Board unanimously authorized the appointment of a new Committee on "Concrete Industrial Houses."

MEETING OF THE BOARD OF DIRECTION, HELD AT THE HEADQUARTERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, NEW YORK, JAN. 16, 1918.

Present: Hatt, Humphrey, Kinney, Turner, Boyer, Wight, Lesley, Alvord.

Minutes of the last meeting of the Board, Oct. 15, 1917, were dispensed with motion, and stood approved.

Treasurer's report showing a balance January 1st $4,139.47 was accepted and approved, and placed in the minutes of the meeting.

After some discussion concerning the expediency of holding the annual convention at the same time and place as in recent years, it was voted that a resolution be prepared by the Board which should be presented to the members of the Institute, to the effect that the convention be postponed until June 24–26, at Atlantic City.

Messrs. Lesley and Wight were appointed to draw up the resolution.

The resolution, approved by the Board, reads as follows:

"*To Members of the American Concrete Institute.* On account of the congestion of the railway service and the expressed wish of the government to restrict passenger traffic to the minimum of necessity at this time, the Board of Direction of the American Concrete Institute on Jan. 16, 1918, decided to postpone the annual meeting of the Institute until June 24–26, 1918, at Atlantic City, N. J.

"Full preparation has been made for the regular meeting at Chicago on Feb. 7, 8 and 9, 1918, but the Directors feel that patriotism impels the postponement until the later date. Copies of the program amended to meet this condition, together with preprints of papers, and reports will be mailed to the members at least two months in advance of the convention. It is hoped that these papers and reports will be made the occasion of written discussion, to be submitted at the meeting in June."

Voted that the preprints be sent to the members at least two months before the convention, also that the question of what papers and reports shall be preprinted be decided by the publication committee.

MEETING OF BOARD OF DIRECTION, HELD AT HOTEL TRAYMORE, ATLANTIC CITY, N. J., JUNE 24, 1918.

Present: Hatt, Alvord, Lesley, Turner, Wason, Humphrey, Boyer, Kinney, Ashton, Wight.

Reading of the minutes of the previous meeting was dispensed with.

The Treasurer's report was accepted and placed on file. This consisted of the report of the Auditors dated June 24, 1918, showing a balance in the bank May 31 of $4378.24 together with a statment from the bank showing June payments of $625.45 and deposited $188.51 leaving a balance June 28, 1918, of $3941.30.

Mr. Humphrey reported that the unpublished portion of Volume XI would cost him more than was anticipated due to the higher cost of printing at the present time. It was voted, however, to publish the same at the earliest opportunity.

Mr. Wight recommended that another year fewer preprints be published. This also seemed to be the consensus of opinion of the Board.

Voted that the President tender the thanks of the Society to the authors of papers presented at the convention.

Voted that the Wason medal be presented in accordance with the terms of the donor covering the past three years and that the President appoint a committee of three to select the respective recipient of the medal.

Voted that H. B. Alvord be elected Secretary for the ensuing year.

Voted that the matter of more permanent headquarters be left to the Executive Committee with power.

Mr. Lesley and Mr. Boyer were appointed on the Finance Committee.

Mr. Kinney and Mr. Turner were appointed on the Executive Committee.

The question as to the place of the next convention was left to the Executive Committee with full power.

A Committee on By-Laws and Constitution was appointed consisting of the President, Mr. Wason and Mr. Lesley.

A resolution concerning the death of Doctor Edgar Marburg was read by Mr. Boyer. This was also accepted by the Board and was read later in the convention.

ATTENDANCE, FOURTEENTH ANNUAL CONVENTION.

D. A. Abrams, Chicago.
B. F. Affleck, Chicago.
E. E. Aiegleres, Franklin, Pa.
R. C. Aldrich, Newark, N. J.
H. B. Allen, Philadelphia.
L. H. Allen, Boston.
H. B. Alvord, Boston.
G. K. Anderson, Philadelphia.
Louis Anderson, Easton, Pa.
Ernest Ashton, Allentown, Pa.

C. F. Barrows, Boston.
P. M. Bates, Pittsburgh.
R. F. Bell, Cleveland.
M. A. Birgert, Washington, D. C.
J. B. Blaw, Atlantic City.
P. A. Boeck, New York.
T. Hugh Boorman, New York.
D. Knickerbocker Boyd, Washington, D. C.
E. D. Boyer, New York.
H. Eltinge Breed, Albany, N. Y.
R. O. Brewer, Philadelphia.
Alexander Brocine, New York.
A. H. Bromley, Canton, O.
E. G. Burkheiser, New York.

A. D. Camp, Long Island City, N. Y.
A. T. Canulette, Slidell, La.
Zenas M. Carter, Cleveland.
C. M. Chapman, New York.
T. W. Clarke, Brookline, Mass.
C. W. Coggeshall, Washington, D. C.
A. B. Cohen, Hoboken, N. J.
P. T. Coons, New York.
H. J. Cox, New York.
V. C. Coxhead, Seattle, Wash.
V. P. Cunningham, Cleveland.
W. A. Curless, Chicago.

Watson Davis, Washington, D. C.
R. C. Delelin, Philadelphia.
R. B. Dickenson, Chicago.
W. A. E. Doying, Washington, D. C.
A. G. Duke, Philadelphia.

J. J. Earley, Washington, D. C.
G. S. Eaton, Chicago.
L. N. Edwards, Toronto.
H. W. Eldridge, New York.

G. N. Francis, Dover, Del.
J. E. Freeman, Chicago.
P. J. Freeman, Pittsburgh.
H. C. Froehling, Richmond, Va.
P. P. Furber, Philadelphia.

Charles Gilman, New York.
H. F. Gonnerman, Philadelphia.
Russell S. Greenman, Albany, N. Y.

L. C. Hanson, Chester, Pa.
R. E. Hanson, Philadelphia.
W. H. Harding, Philadelphia.
O. C. Hasse, Wampum, Pa.
W. K. Hatt, Lafayette, Ind.
L. C. Hawk, Nazareth, Pa.
Rudolph Hering, New York.
M. E. Hibbs, Philadelphia.
A. M. Hirsh, Ampere, N. J.
S. C. Hollister, Philadelphia.
W. A. Holman, New York.
A. S. Hook, Chicago.
N. S. Hough, Pittsburgh.
R. H. Hughes, Wampum, Pa.
E. C. Hulbert, Wampum, Pa.
Walter A. Hull, Pittsburgh.
Richard L. Humphrey, Philadelphia.

A. N. Johnson, Chicago.
N. C. Johnson, New York.

F. W. Kelley, Albany, N. Y.
J. E. King, Philadelphia.

L. H. Kinney, Philadelphia.
W. M. Kinney, Chicago.
F. A. Kirkpatrick, Pittsburgh.
H. M. Knight, New York.

Simon Lake, Milford, Conn.
E. L. Lasier, Washington, D. C.
R. W. Lesley, Philadelphia.
W. C. Lewis, Washington, D. C.
J. H. Libberton, Chicago.
E. Lichtenberg, Milwaukee.
A. E. Lindau, Buffalo.
L. L. Livingston, New York.
F. K. Lord, New York.
H. J. Love, Cleveland.
J. W. Lowell, Pittsburgh.

J. E. McAfee, Boston.
F. R. McMillan, Philadelphia.
H. F. McMullen, Haddon Heights, N. J.
E. S. Martin, Philadelphia.
C. W. Mayers, Boston.
A. J. Maynard, State Farm, Mass.
E. J. Mehren, New York.
Coleman Meriwether, Chicago.
J. G. Mingle, Cleveland.
M. D. Morrill, Claymont, Del.
C. M. Morssen, Montreal.
Albert Moyer, New York.
Roy Mullins, Camden, N. J.
T. D. Mylrea, Toronto.

R. C. Narrin, New York.
Paul Negrier, Washington, D. C.
W. L. V. Nichol, Washington, D. C.
Thomas Nolan, Philadelphia.

J. C. Olson, New Orleans, La.
J. B. Orr, Miami, Fla.

J. C. Pearson, Washington, D. C.
E. G. Perrot, Philadelphia.
J. P. H. Perry, New York.
O. R. Perry, Philadelphia.
A. Polk, Wilmington, Del.
F. A. Randall, New York.
A. S. Reed, Wilmington, Del.
W. P. Reed, Boston, Mass.
W. E. Rew, Norfolk, Va.
George Rhines, New York.
F. E. Richart, Philadelphia.
G. A. Ricker, Washington, D. C.
W. G. Roberts, Boston, Mass.
H. D. Robbins, Trenton, N. J.
Sydney Rogow, Cleveland.

E. B. Sadtler, New York.
T. B. Schertzer, New York.
Adolf Schiller, Haddon Heights, N. J.
W. H. Schouler, Newark, N. J.
H. M. Scofield, Philadelphia.
J. W. Scull, Atlantic City.
R. L. Shainwald, New York.
W. A. Slater, Philadelphia.
B. S. Smith, Philadelphia.
Edward Smulski, New York.
A. V. Spinosa, Pittsburgh.
E. A. Steele, Philadelphia.
H. S. Sterling, Philadelphia.

A. N. Talbot, Urbana, Ill.
W. H. Thompson, Buffalo.
F. A. Track, Washington, D. C.
H. C. Turner, New York.
R. K. Turner, Springfield, Mass.

W. L. Umstad, Pittsburgh.
J. S. Unger, Pittsburgh.
C. M. Upham, Dover, Del.

H. S. Van Scoyoe, Montreal.

B. H. Wait, New York.
Stanton Walker, Chicago.
H. H. Ward, New York.
M. B. Warner, New York.
C. L. Warwick, Philadelphia.
L. C. Wason, Boston.
O. E. Wasson, Hudson, N. Y.
E. J. Waterweich, New York.
C. D. Watson, Wycombe, Pa.

W. J. Watson, Cleveland.
H. M. Westergaard, Philadelphia.
A. D. Whipple, Chicago.
Harvey Whipple, Detroit.
F. R. White, Washington, D. C.
W. E. White, Springfield, Mass.
R. J. Wig, Philadelphia.
F. C. Wight, New York.
T. A. Williams, Camden, N. J.
I. H. Woolson, New York.
J. S. Wormeley, Washington, D. C.
R. B. Young, Toronto.
J. L. Zeidler, St. Joseph, Mo.

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